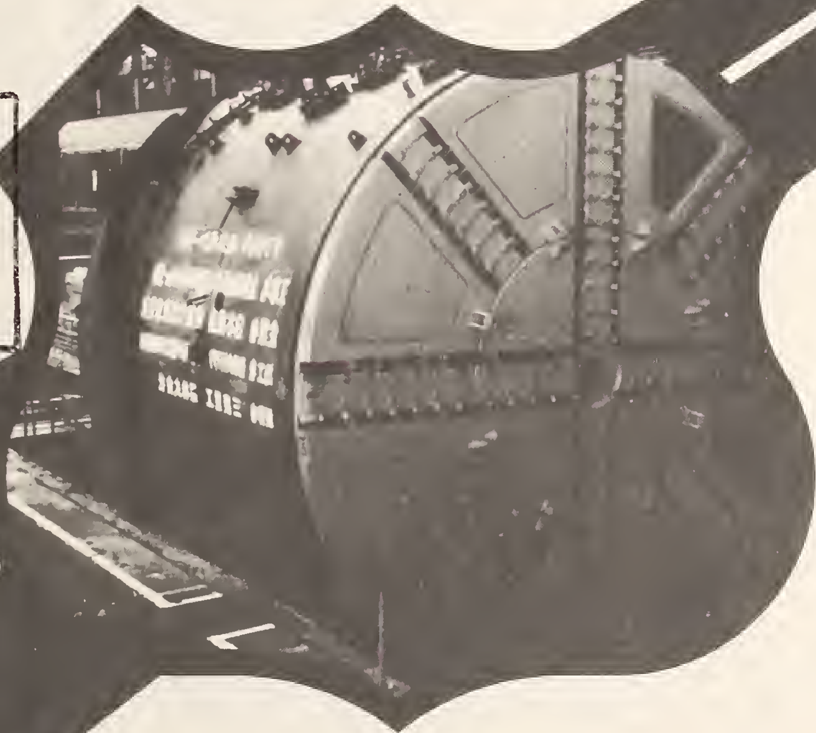
). FHWA/RD-81/073

NDWATER CONTROL IN TUNNELING

Vol. 1. Groundwater Control Systems for Urban Tunneling
April 1982
Final Report



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Federal Highway Administration

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Structures and Applied Mechanics Division
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FOREWORD

This three volume report summarizes best available practices in groundwater control both during and after tunnel construction. This volume describes typical problems encountered during construction, recommends site exploration procedures needed to define groundwater problems, describes alternate methods for groundwater control, establishes selection criteria, and touches on legal and contractual considerations.

Sufficient copies of the report are being distributed to provide two copies to each regional office, one copy to each division office, and two copies to each State highway agency. Direct distribution is being made to the division offices.

Charles F. Scheffey
Director, Office of Research
Federal Highway Administration

NOTICE

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16. Abstract

This volume is a summary of current practice on groundwater control systems used during construction of both cut-and-cover and bored tunnels in urban areas. Eight (8) groundwater control methods are discussed including dewatering, recharge, cutoff walls and trenches, grouting, freezing, compressed air, slurry and earth pressure balance shields and electro-osmosis.

Method selection procedures, contractual and risk considerations and cost factors are also discussed.

Three companion volumes include:

Volume 2: Preventing Groundwater Intrusion into Completed Transportation Tunnels (FHWA/RD-81/074)

Volume 3: Recommended Practice (FHWA/RD-81/075)

Executive Summary (FHWA/RD-81/076)

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PREFACE

This is Volume 1 of a three-volume series devoted to groundwater control during and after construction of transportation tunnels in urban areas. The work presented herein is a compilation of direct experience of the principal authors, along with that of selected contractors and consultants familiar with groundwater control methods.

Volume 1 discusses methods for controlling groundwater during construction of tunnels.

Volume 2 discusses methods for controlling groundwater during the life of the tunnel.

Volume 3 is a compilation of Volumes 1 and 2 into a concise format, including example problems plus recommendations for future investigations.

The work has been sponsored by the Federal Highway Administration of the U.S. Department of Transportation, Office of Research Structures and Applied Mechanics Division.

Individuals outside of the performing organizations who gave freely of their time and expertise included:

Mr. Robert S. Mayo	-	Robert S. Mayo & Associates, Washington, D.C.
Mr. Richard Redmond	-	MacLean Grove & Company, Greenwich, Connecticut
Mr. Joseph Bhore	-	Perini Corporation, Framingham, Massachusetts
Mr. John A. Shuster	-	Geofreeze, Lorton, Virginia
Mr. Robert M. Snyder	-	Tekken Construction Co. Ltd., Walnut Creek, California
Mr. Joseph Welch	-	Hayward Baker Co., Odenton, Maryland

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APPLICABLE SI UNIT CONVERSIONS

AREA

1 acre	= 4047 sq. m.
1 sq. in.	= 6.45 sq. cm.
1 sq. ft.	= 0.0929 sq. m.
1 sq. mi.	= 2.59 sq. km.

DENSITY

1 lb. mass/cu. ft.	= 16.018 kg/cu. m.

FLOW

1 gallon/min.	= 0.063 l./sec.
1 gallon/min.	= 0.00379 cu. m./min.

FORCE

1 lb. - force	= 4.448 Newtons
1 kg. - force	= 9.807 Newtons

LENGTH

1 in.	= 25.4 mm.
1 ft.	= 0.3048 m.
1 yd.	= 0.9144 m.
1 mi.	= 1.609 km.
1 mil	= 0.0254 mm.

PRESSURE AND STRESS

1 lb. per sq. in.	= 6.895×10^{3} Pa.
1 atm.	= 1.013×10^{5} Pa.
1 kg-force/sq. cm.	= 9.807×10^{4} Pa.
1 bar	= 1×10^{5} Pa.
1 kip/sq. in.	= 6.895×10^{6} Pa.
1 lb-force/sq. ft.	= 47.88 Pa.

VOLUME

1 cu. in.	= 16.4 cu. cm.
1 cu. ft.	= 0.0283 cu. m.
1 cu. yd.	= 0.765 cu. m.
1 gal.	= 0.00379 cu. m.

1.00 INTRODUCTION

1.10 OVERVIEW

Groundwater control is probably the single most significant geotechnical variable to be dealt with during tunnel construction. To quote Schmidt (Ref. 234),

> "More tunneling problems and hazards are associated with the appearance of groundwater than with any other single factor."

and Bartlett (Ref. 42),

> "Subterranean water remains the tunneler's worst enemy."

The process of tunnel excavation is uniquely susceptible to groundwater difficulties. In open cut excavation, a problem with groundwater control along part of the alignment usually causes a shift of excavating activities to some other section while the groundwater problems are being diagnosed and remedied. However, in tunneling, all of the production is concentrated on excavation at the tunnel heading. If groundwater problems stop or diminish the efficiency of excavation at the heading, the entire tunnel construction operation is affected in direct proportion; there is no alternative focus of construction activities while groundwater problems are resolved.

Nearly all ground stability problems, whether in soil or rock, are related to, or controlled by, groundwater inflows. They are typically less severe in rock tunnels than in soil tunnels. Provided adequate pumping capacity is available, rock tunnels can usually be dewatered and advance continued with less difficulty than in soft ground. A primary concern is that localized flows in rock can be very large and numerous examples of tunnel floods resulting in loss of life and equipment can be found in the literature. Methods available for investigation and prediction of subsurface conditions and for identification of potential groundwater problems are discussed in Section 3.00.

In soft ground tunneling, the problems are typically associated with ground stability. Tunnels driven through saturated sands and silts require groundwater control measures to prevent ground runs and possible surface settlement. Dewatering, cutoffs, grouting, freezing, compressed air, slurry shields, and earth pressure balance shields are all possible construction measures which can be considered and are discussed in Section 4.00.

Compounding the major problems that groundwater can create during tunnel construction is the unfortunate fact that the state-of-the-art of quantitative predictions of groundwater behavior is imprecise at best. Soil and rock permeabilities typically vary over eight to ten orders of magnitude, whereas the range in other soil properties, i.e., compressibility, shear strength, density, etc. is typically less than one order of magnitude. While the state-of-the-art of groundwater control has advanced rapidly in recent years, for example in the fields of designing, constructing, and operating multiple well dewatering systems and chemical grouting technology, there are still fundamental barriers to the precise prediction of how the geohydrologic environment will respond to a specific groundwater control technique. These barriers are due primarily to inadequate subsurface and hydrologic data, a result of routine exploration programs which generally ignore geohydrologic details. It is often said that soil and rock are the most variable materials of construction; their hydraulic properties are the widest ranging, least predictable, and are the most likely to vary significantly over relatively short distances. Because all tunneling activity is focused intensely on excavation at the heading, and because this excavation is profoundly influenced by the presence of groundwater, these concepts of variability and predictability of groundwater conditions become basic determinants of the cost of tunneling. State-of-the-art skill is imperative in controlling the costs of groundwater control, but experienced engineering judgment is indispensable in achieving satisfactory results. Section 5.00 is a discussion of criteria for selection of groundwater control methods.

Due to uncertainties associated with groundwater control, serious legal and contractual dilemmas arise that require open communication for proper solution. These uncertainties and their major impact on construction costs result in serious questions of risk allocation. Who bears the risk? The owner, the engineer, or the contractor? How should the contract be structured to properly allocate the risk? This key matter is probably as important to proper groundwater control as technical issues. A discussion of legal and contractual considerations is presented in Section 6.00.

1.20 PURPOSE

The purpose of this report is to present an overview of groundwater control systems available for use during the construction of cut-and-cover tunnels, soft ground tunnels, and hard rock tunnels in urban areas. The report is based primarily on experience in the United States, however, western European and Japanese experience has been considered, primarily in connection with highly specialized techniques. It is not a "how to" report, but is rather an overview directed at planners, designers, and builders of transportation facilities in urban areas to give

them an informed appreciation of groundwater control methods and the impact they can have on a project. There are three companion volumes to this report including:

- Volume 2. "Preventing Groundwater Intrusion Into Completed Transportation Tunnels." A state-of-the-art report directed at control of groundwater in completed tunnels, and

- Volume 3. Recommended Practice "A manual containing case specific information on control of groundwater both during construction and in completed tunnels.

- Executive Summary

Considerable work has been done by a diverse group of individuals and organizations concerning the problem of groundwater control in tunneling as is evidenced by the more that 280 references included in this volume. A similar body of information also exists in non-English publications, but is not included.

1.30 SCOPE

Preparation of this volume has included searching for authoritative information, studying the literature, consulting experts, visiting job sites, synthesizing the information, and developing guidelines for design and operation of dewatering systems. The work has included the exercise of experienced judgment in the synthesis of current and best practices.

Sources of information consulted under this contract have included:

1. Direct experience by personnel of Goldberg-Zoino & Associates, Inc., Ground/Water Technology, Inc., and Jacobs Associates.

2. The files of these three firms relative to actual tunnel projects.

3. Records of interview by correspondence and in person, with contractors, engineering firms, public agencies, and individual consultants.

4. Published literature.

Major tasks completed include:

1. Summarizing problems attributable to inadequate groundwater control. (Section 2.00)

2. Descriptions of procedures for evaluation of subsurface conditions at and in environs of the site. (Section 3.00)

3. Description of current best design and construction practices. (Section 4.00)

4. Development of criteria for selection of groundwater control methods. (Section 5.00)

5. Description of best current technical, contractual and legal practice. (Section 6.00)

2.00 PROBLEMS ASSOCIATED WITH INADEQUATE GROUNDWATER CONTROL

2.10 GENERAL

Soft ground tunneling is typically more susceptible to problems associated with inadequate groundwater control than rock tunnels. Without groundwater control, otherwise manageable ground can become unstable resulting in soil and water flows that, in the extreme, can bury the tunneling equipment. Groundwater gradients towards the tunnel can cause quick conditions to occur. When an aquifer is penetrated by tunnel excavation, a sudden inflow of water may carry large quantities of soil into the tunnel. A chimney may be created to the ground surface or widespread surficial settlements may occur.

Although tunnels in soil are typically more troublesome, major problems can occur due to groundwater inflows during construction of rock tunnels, as well. Even small quantities of water can lubricate joints or shear planes, causing decreased ground stability. Small quantities of water under pressure can wash material from gouge zones causing equipment problems as well as ground instability. Groundwater inflows in rock tunnels can be extremely erratic with flows occurring in highly localized zones. If undetected, such zones can flood transit sized tunnels.

A less common problem can occur when tunneling through water-sensitive materials such as expansive shales, bentonite and anhidrite. Even where natural groundwater is not present, construction water can be sufficient to cause swelling problems.

2.20 DESIGN PROBLEMS

While there are, in fact, relatively few design problems that can be said to be associated with inadequate dewatering, the design of a project must take dewatering considerations into account. For instance, the investigation program must have as a major objective the evaluation of groundwater conditions. The importance of accurately performing and interpreting a test boring and groundwater observation program cannot be stressed too highly.

2.21 Soft Ground Tunneling

A case in point concerns a sewer tunnel in New York City. The pre-bid information led contractors to believe that the entire tunnel was to be constructed in glacial till. An ejector dewatering system was designed to handle the anticipated minor seepage from the relatively impervious till. When a far greater amount of water was encountered, the dewatering contractor had

to stop and re-design the system. Careful study of additional data revealed an underlying stratum of outwash sand and gravel, which was significantly different from the upper glacial till through which the tunnel was being driven. Once the actual situation was revealed, a few deep wells were installed into the sand and gravel to augment the ejector system. This combined system adequately controlled groundwater on the job.

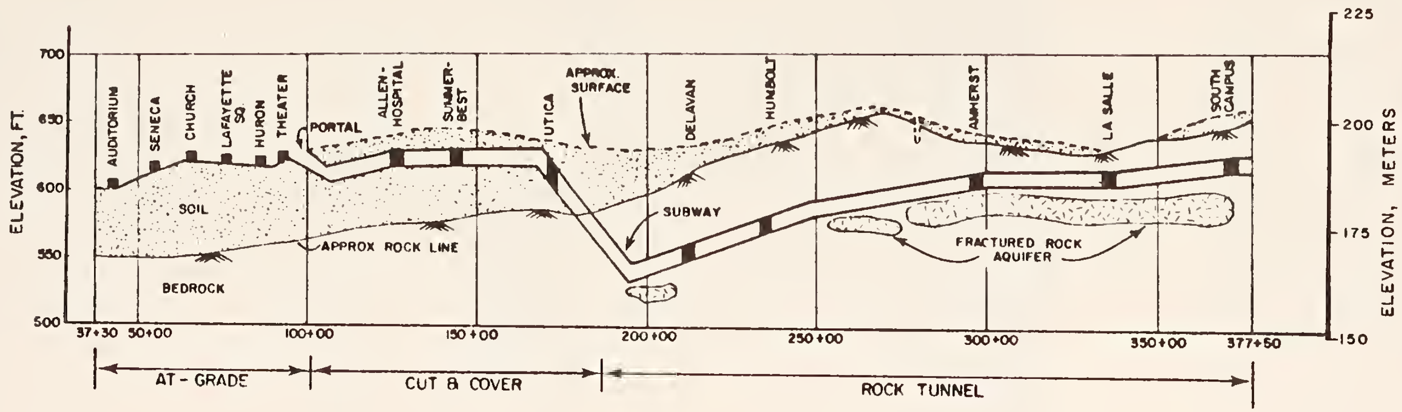
Once initial investigations have been completed, the information should be reviewed with respect to alignment and tunneling methods and determination of the possible impact of groundwater conditions on these considerations should be made. In certain cases, a minor realignment can save considerable time and money and minimize groundwater problems. Most tunnel machines are designed for use under specific conditions and when unanticipated conditions are encountered major problems can result.

2.22 Rock Tunneling

A recent project on which thorough study of groundwater conditions lead to re-design of the tunnel alignment is the rock tunnel section of the Light Rail Rapid Transit Project in Buffalo, New York, presently under construction (Figure 1). The tunnels were originally designed at an elevation where they would have intercepted a confined aquifer of highly fractured rock which was undetected by the preliminary investigation program. The aquifer had a transmissibility of approximately 90,000 square feet per day (8361 square metres per day) and could have resulted in flooding of the tunnel during construction if it were not adequately identified during the design phase investigations. The intensive investigations, which included many test borings, borehole permeability tests, test shafts, and long-term pumping tests, resulted in a realignment of the vertical profile to avoid this potentially serious groundwater problem. The problem was of sufficient magnitude that at one time the feasibility of the project was considered to be in question. However, based on geologic data and pumping test results, it was concluded that a properly designed dewatering system could reduce hydrostatic pressures in this aquifer to safe levels below tunnel invert and that the tunnels could be driven safely without major groundwater inflows.

2.23 Mixed Face Tunneling

Mixed face tunnels present some of the greatest challenges to be met by tunnel designers and builders. Groundwater conditions at the soil/rock interface are frequently difficult to control. A portion of twin tunnel line along the Red Line extension of the Massachusetts Bay Transportation Authority in Somerville, Massachusetts, is a case for consideration (Figure 2). The tunnels go through a transition from rock tunnel to full face



PROFILE

FIGURE 1 - Profile of Buffalo LRRT Project

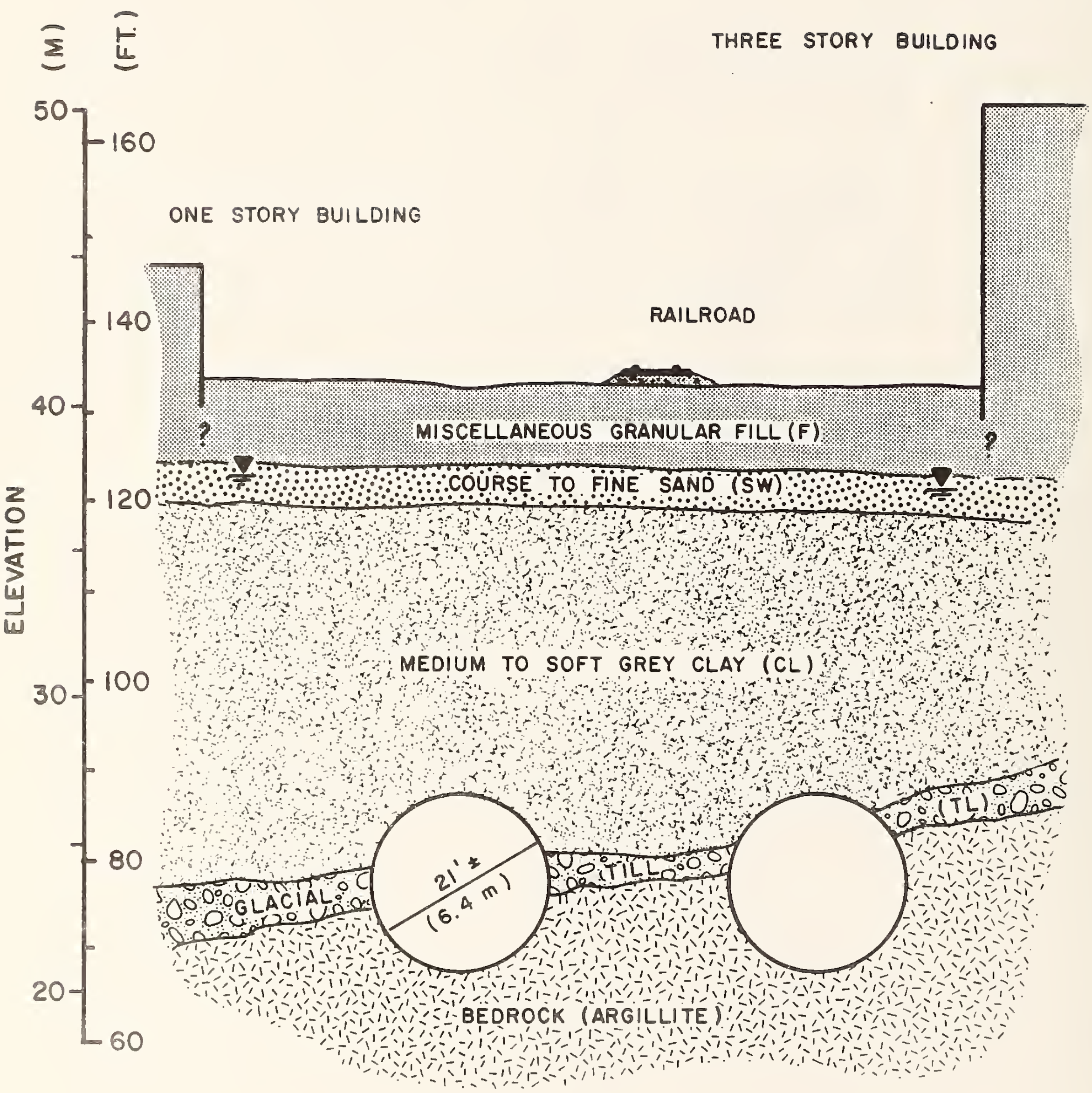


FIGURE 2 - Geologic Section, Davis Square to Porter Square Subway Tunnels, Somerville, Massachusetts

in marine clay. Between the bedrock and clay is a 3 foot (1m) thick stratum of glacial till which is extremely heterogeneous and varies in composition from clean sand to gravelly clay. In order to control potentially large groundwater inflows from the fractured bedrock and locally pervious till, a multiphase grouting program was required in advance of tunnel excavation. The program included cement grouting of both the bedrock and glacial till. Compaction grouting was also implemented to minimize surficial settlement of an adjacent brick building. The tunnel has been excavated with minimal difficulty under conditions which, if not fully appreciated ahead of time, could have been disastrous.

2.24 Surface Access Considerations

Tunnel design must take into account anticipated construction problems both within the tunnel and at the surface. Many groundwater control methods require considerable surface access and this must be considered during the design process. For instance, although some tunnels may be designed to be driven under compressed air, it is often necessary to use some other groundwater control method for construction of the shafts and installation of air locks. Surface access was a problem in connection with construction of a portion of the Baltimore, Maryland Subway System. Tunnels were designed to be constructed under compressed air; however, access shafts required pre-drainage by wells for a distance of 200 feet (61.0m) away from the shaft in order to permit setting of locks and tunnel shield in the dry. This was not possible because the section in which the shield and lock were to be installed fell directly beneath a parking garage and there was no access for installation of dewatering wells. Fortunately, the soils were not as permeable as had been anticipated and existing wells at the shaft and adjacent to the garage controlled the groundwater sufficiently to permit installation of the compressed air lock.

2.30 CONSTRUCTION PROBLEMS

Tunneling differs from other types of construction because all productive activity is concentrated at one location - the heading. If problems develop at the heading, productivity is lost; there is no other work for the excavating crew to do, and therefore money and time are wasted until the problem is corrected. Other forms of construction, such as cut-and-cover or open excavation, are more flexible because problems in one location can be isolated while work can proceed at others. In a tunnel, the heading problem must be solved before tunnel advance can proceed. The costs of dealing with unexpected problems at the time of tunnel excavation are usually greater than those

costs to permit a preconstruction recognition of the problem. Once an unanticipated problem is encountered, it is usually more cost effective to overdesign remedial measures, thereby assuring resumption of production.

Tunnel production is dependent on many variables associated with excavation and muck removal, however, stability of the tunnel crown and face is probably the single most significant variable affecting production rates. Typically, soil or rock stability is related to groundwater conditions. Problems can vary from nuisance value to disastrous in which inflows cause flooding or major instability.

Proper groundwater control in both cut-and-cover and soft ground tunnels also has a major influence on ground movements adjacent to and directly above the tunnel. Dr. H. Q. Golder made the following observation in 1971 (Ref. 111):

> "It is astonishing to many inexperienced engineers, that the first time they meet the phenomenon, how far the influence of bad or unsafe construction methods can extend from the source of the operation, e.g., many contractors will attempt to dig in fine sand below water - convinced that they can bull through. They can't. And cracking of any building within a radius of 100 yards (91.4m) or more can rightly be attributed to their operations."

2.31 Running Ground

The extent of problems caused by seepage usually depends upon the type of soil and seepage forces. Uniform fine sand or non-plastic silt will "run" under minor seepage forces, while a well graded, coarse sand or gravel will not.

Running ground occurred during construction of the Kinki Subway in Osaka, Japan. The material at the face was fine grained with an overlying layer of coarse sand. Deep wells did not completely drain the fine material, and until an ejector system was tried for predrainage, running sand at the face slowed production and also caused a collapse to the surface in one area.

Running ground problems also occurred on a section of the Washington Metro system. The complex soil stratification resulted in frequent instances of mixed face conditions. Widely spaced deep wells were incapable of completely draining the stratified soils. Although an area was grouted from the surface, minor flows of water through small layers of permeable soil caused running ground and daylighting occurred in a local area.

2.32 Subsidence

Inadequate groundwater control methods can result in surface subsidence due to lowering of groundwater levels outside the limit of either open excavation or tunnels because of increased effective stresses on compressible soils. One of the most significant projects where consolidation of surrounding soils was a potentially major problem was in San Francisco. During construction of the BART system there was considerable concern over potential damage to nearby facilities due to compression of soft bay mud outside of dewatered cut-and-cover tunnel excavations. Prior to construction it appeared that groundwater recharge systems would be required to prevent consolidation. However, the need for these expensive systems could not be determined with complete confidence. The implication of this uncertainty on bidding was predicted to be very large and therefore a contractual form was developed to remove the contingency from the bidding process. It was decided to allow for a cost plus payment method if, in the opinion of the engineers, a recharge system should be installed. No recharge system was required resulting in substantial cost savings.

2.33 Release of Dissolved Gases

Other problems associated with groundwater control during construction include the release of dissolved gases in groundwater. These gases typically include hydrogen sulphide and methane, and problems of this type are reported in several cities including Los Angeles, Detroit, Rochester and Buffalo. On the Buffalo LRRT Project, groundwater is treated with hydrogen peroxide to remove hydrogen sulphide from the water prior to disposal into surface water bodies.

2.34 Compressed Air Pressures

In compressed air tunneling, a common problem is determination of the proper air pressure to keep the face dry and stable. Air pressure designed to completely prevent groundwater inflow may be too high at the arch causing blowouts and major air losses. Pressure designed to maintain a dry arch may be insufficient to prevent water from entering at the invert. In addition to escalating labor costs due to shorter required working shifts, high air pressure is also detrimental to health. As a result of these problems, innovative soft ground tunneling shields have been developed in Europe and Japan which eliminate the need for compressed air by maintaining face stability and controlling groundwater through the use of either bentonite slurry or earth pressure maintained at the shield face. These newer soft ground shields are described in Section 4.80.

2.35 Rock Tunneling

Groundwater control in rock tunnels is usually not as great a problem as in soft ground tunnels because rock is typically less sensitive to the presence of water. Tunneling can normally proceed if sufficient pumping capacity is available within the tunnel or proper drainage has been provided. The major problem associated with groundwater in rock tunnels is reduction of shear strength along joints or removal of joint filler materials that can cause base or crown instability that must be controlled by use of steel ribs, rock bolts, or shotcrete. Large groundwater flows into a sewer tunnel excavated by drill and blast methods in Buffalo, New York was so great that the contractor was unable to place concrete without significant loss of fine aggregate and cement paste. Cement grouting was attempted, but flows were too great to permit the grout to form an effective seal. After several unsuccessful grouting attempts, an extensive system of groundwater diversion pans and form drains was developed which allowed the contractor to place the lining successfully.

2.40 CONTRACTUAL INADEQUACIES

Frequently, more than one groundwater control technique is feasible for application to a particular tunnel section. However, method selection and designer selection are two issues that can lead to project difficulties.

While it appears that design of the groundwater control system by the Owner's engineer limits bidding contingencies, there are numerous disadvantages to this approach. The contractor loses the opportunity to apply his ingenuity to the selection of a dewatering method compatible with the alternative construction techniques. The system design is based on the limited pre-bid information and it is difficult to retain the flexibility necessary to adapt to changed conditions. With a complete Owner designed system, the Owner's responsibilty extends to proper installation and operation. Quality control is difficult because inspectors experienced in dewatering work may not be available.

The chief difficulty in the Owner-designed system is the unavoidable confusion over responsibility. With the Owner fully responsible for control of groundwater, he is directly involved in the contractor's productivity. As a result, the contractor is in a position where he no longer has complete control of his operations and the stage is set for confusion, disputes, and additional costs.

A more effective alternative to complete design of the system is for the Owner to specify only the result the system is to produce, together with the procedures that will be used to measure this result. This approach gives the contractor

complete freedom in selection of the techniques to be used. It does not intrude upon the contractor's control of the work, and places the full responsibility for control of groundwater on him. This approach has proven to be effective in concentrating the necessary skill and motivation on the part of the people directly responsible for control of groundwater. For a more thorough discussion of contractual details refer to Section 6.20.

The degree to which the groundwater control method is specified should depend upon anticipated difficulties. If the materials are highly variable, a flexible design is needed. In such a case a single specified design is highly liable against claims for improper performance. Advantages of a flexible groundwater control system are discussed in Section 3.70.

If the Owner does not choose to specify a groundwater control method, there should be complete disclosure of all subsurface information as part of the bidding documents. This will allow the design of groundwater control methods to be as competitive as possible. In such cases, bidding contingencies can be reduced through the provision of paid extras. For example, if a contract was specified to be built in the dry (requiring pre-drainage), a pay item, which will reimburse the contractor if recharge is seen to be necessary, will reduce bidding contingencies. This approach was used successfully during construction of the BART System in San Francisco.

The amount of design and specification of a groundwater control system must therefore be viewed in the context of a sharing of risk between the Owner and the Contractor with the degree of sharing having a major effect on bid prices.

2.50 EFFECTS ON PROPERTY AND PEOPLE

Effects on property and people outside of the immediate construction area caused by groundwater control operations can be significant. They are typically related to ground movements resulting from soil consolidation or loss of ground.

The potential for structural damage related to dewatering is centered primarily about settlement. Settlement occurs due to either one or both of two basic mechanisms. The first is removal of soil due to erosion by the removal of fines by pumping through inadequate filters or poor well construction. The second is due to consolidation of cohesive soil as a result of increased load due to reduced groundwater levels. An example of concern for such a mechanism was discussed in Section 2.32 in connection with the BART project.

Ground movement and distress associated with hard rock tunneling is typically not a problem except in the cases where groundwater levels are reduced and overlying soils consolidate as a result of drainage into the tunnel. It is not a common problem and therefore is frequently ignored, which is a risky course of action. The soils overlying all rock tunnels must be evaluated for potential consolidation due to lowered groundwater levels.

Wooden piles, timbers, or cribbing may deteriorate when exposed to air during groundwater lowering. Such concerns frequently lead to the design of groundwater recharge systems to maintain groundwater levels around untreated wood elements of foundation structures. This was done for the supporting anchors for the Brooklyn Bridge, which was protected by recharge systems during construction of a nearby interceptor sewer tunnel. In Boston, a major foundation excavation was surrounded by buildings supported on wood piles and a large recharge system was operated to maintain groundwater levels outside of this excavation.

Settlement due to consolidation of fine grained soils is time dependent. If effective stress increases caused by dewatering can be limited to relatively short times as compared to the time required for significant consolidation to occur, then dewatering may be permitted without causing distress to surrounding areas. For instance, because of anticipated settlement problems, no dewatering was to be allowed during construction of a major sewer tunnel in New York City. Initial attempts to drive the tunnel under compressed air caused severe difficulties with loss of air to the street, with surface heaving reported. Later attempts to freeze sections of the tunnel, first with freeze walls parallel to the alignment, and secondly by freezing the face, produce mixed results. When the expected completion date was a year behind schedule, permission was granted to dewater a test section to allow the tunnel to be driven under reduced air pressure. The test was successful and the tunnel was completed under low air pressure with partial dewatering. There was no evidence of settlement caused by the dewatering operation as water levels were depressed for a relatively short period when compared to the time required for consolidation of the fine grained soils.

Catastrophic loss of the face and subsequent piping, erosion and daylighting produce serious safety hazards to workmen and the general public from soft ground tunneling. Failures of this type are typically associated with fine grained cohesionless soils subjected to an excessive inward flow gradient at the face.

Economic harm to third parties can result from groundwater control operations as well as other phases of urban construction. The most obvious harms are disruption of traffic and interruption of normal business, and possible structural damage from settlement. A less obvious effect is the potential depletion or contamination of the groundwater resource due to extensive pumping. In special instances where recharge is necessary, the groundwater level may be raised above the original waterproofing line of adjacent underground structures causing flooding of basements and utilities. In some instances, ponding and icing may occur at ground surface. In rare instances, dewatering system discharge may create problems of its own; the flow rates may stress the capacity of storm or combined sewer systems and the chemical constituents of the water (such as dissolved iron on hydrogen sulfide) may overload sewage treatment facilities or increase treatment costs.

2.60 ENVIRONMENTAL EFFECTS

The quantity and quality of the discharged water can cause substantial problems and dramatically affect cost, as has been the case for disposal of up to 16,000 gpm (960 l/s) from tunnel and open cut excavations for the Buffalo LRRT System. The groundwater contains up to 3 ppm of dissolved hydrogen sulphide. The existing sewer system can not handle the additional volume and the municipal treatment facilities are not designed to accept high concentrations of hydrogen sulphide. A separate treatment plant and 3.5 mile (5.6km) long 18" to 36" (46 to 91cm) diameter sewer has therefore been designed and built to collect and treat the pumped water prior to final discharge to a nearby stream.

The removal of water from an aquifer along the coastline or adjacent to tidal marshes or rivers can lead to saltwater intrusion into the aquifer. This potential needs to be recognized in the preliminary investigation stage in order to properly evaluate the need for cutoff walls or groundwater recharge systems. It will also affect the choice of dewatering method and the approach taken in specifying and monitoring performance.

A less obvious potential contamination source is leaky sewers. A leaky sewer located below the groundwater surface, acts as a drain. It receives inflow from the groundwater and prevents outflow of waste water. However, if groundwater levels are lowered below the sewer, the seepage is reversed allowing waste water to enter the aquifer. If such a problem is not detected until dewatering is in progress, third party claims, and political ramifications can be substantial.

Cutoff walls or groundwater recharge may be required if a heavily used aquifer will be depleted. Alternatively, water from other sources may have to be provided at the Owner's expense. This occurred in New York City, where it was anticipated that, an aquifer would be depleted and saltwater intrusion could result due to combined pumping for dewatering and for water supply. The water company therefore agreed to shut off their wells near the construction site in return for an equal quantity of water supplied by the Owner.

Aesthetic effects may also become a factor in design such as occurred during construction of a depressed roadway near the Capitol in Washington, D.C. Due to dewatering for the excavation, it was feared that the trees adjacent to the right-of-way would die. A recharge system was installed to prevent the destruction of the trees. Similar concerns were also expressed for trees on the Harvard University campus adjacent to the Harvard Square station of the MBTA Red Line in Boston.

Another example of aesthetic effects occurred in Buffalo, New York where deep tunnel dewatering has resulted in depletion of inflow into a spring fed lake within a local cemetery. As a result, the level of the lake has dropped several feet and a lining and recharge scheme has been developed for use, if necessary, to maintain an acceptable lake level.

2.70 SUMMARY

From all of the foregoing one can see that problems associated with inadequate groundwater control involve:

-- Design

-- Construction

-- Contractual Relations

-- Effects on property and people

-- Effects on the environment

Design practice in the United States is such that groundwater control considerations influence design in only those major cases where location of a specific aquifer may influence route selection. However, in the years to come as chemical grouting and freezing gain greater acceptance by designers it is believed that designers will incorporate their use as part of the design.

Effects of inadequate groundwater control on construction are numerous and varied. Nearly all production difficulties in both soft ground and rock tunnels are related in some way to groundwater conditions.

Improper definition of groundwater control responsibility in contractual documents will almost certainly result in improper control and ultimate litigation. Documents need to specify a desired result and permit maximum flexibility during construction for modification of control methods as project experience is gained.

Effects on property and people typically result from ground movement due to consolidation and loss of ground. Other significant effects include the physical presence of control systems in the form of pipes, pumps, compressors, etc.

Environmental considerations are becoming more and more restrictive. Disposal of water can no longer be done without careful study of its quality and comparison with the quality of receiving waters. In urban areas, the noise associated with operation of pumps and compressors is an environmental consideration of increased concern.

3.00 EVALUATION OF SUBSURFACE CONDITIONS

3.10 GENERAL

Explorations undertaken to help evaluate groundwater conditions are commonly of secondary importance to other exploration programs. Soil and rock parameters such as shear strength, gradation, density, compressibility, in-situ stress, and joint characteristics are thoroughly investigated while important geohydrologic parameters are given inadequate consideration.

In developing a subsurface exploration program one must recognize from the outset that geohydrologic conditions may be critical to the successful completion of the project. An early preliminary evaluation of transmissivity, storage capacity, geohydrologic boundary conditions and water quality will identify potential problems. Such problems are almost always easier to address in the early planning stages than during latter design stages. While this is true of most engineering problems, it is particularly true of groundwater problems which, if unforseen, will, as a minimum, result in project delay.

Preliminary studies must always involve study of the regional geology. Most of the investigation is typically concentrated in the immediate vicinity of the tunnel, however, the regional geology is also important for evaluation of groundwater control measures. The continuity and extent of pervious soil zones, fissured rock and solution cavities and their relation to recharge sources cannot be overlooked, especially where these zones are encountered intermittently along the tunnel alignment. Such data may falsely assure designers and contractors that groundwater control will be a minor consideration.

As discussed in Section 3.51, the effective permeability of a geologic unit can best be evaluated by large scale field testing. The permeability of soil can vary by eight to ten orders of magnitude, with major variations being accounted for by small changes in the percentage of fines. The problem is further complicated because soil and rock aquifers are often highly anisotropic. The permeability in the horizontal direction is frequently at least two orders of magnitude higher than the permeability in the vertical direction. While laboratory testing is useful, it should not be relied upon as the sole indicator of permeability.

A major purpose of subsurface hydrological investigations is determination of the project feasibility with respect to the present state of knowledge of subsurface conditions before excavation starts. Type and scope of problems must be identified well in advance of construction to minimize construction contin-

gencies. The tunnel excavation is always on the critical path and therefore, any delay at the heading will be directly related to increases in cost. The investigation generates basic data for design and cost estimating and adds to the understanding of the social, economic, and environmental effects of the project.

The investigations should produce several parameters:

1. The permeability of the formation as an indicator of anticipated water inflow, grout take, etc.

2. The gradation or grain size distribution of cohesionless soils as an indicator of permeability. In addition, it is an indicator of susceptibility of the material to piping or erosion.

3. The density of granular material is an indication of the stability of the material. Loose soils are potentially more unstable than dense soils in the presence of water.

4. The compressibility of cohesive soils is a measure of the potential for surface subsidence associated with dewatering.

5. The aquifer geometry (thickness and extent, relation to barrier boundaries and recharge sources) is of importance in determining the extent of influence, time of response, and required pumping rates to maintain the desired drawdown for dewatering or recharge systems.

6. Water chemistry parameters as indicators of the potential of the groundwater to corrode or encrust pumping equipment, or to affect grout or bentonite slurry in-situ, and to create possible disposal problems.

7. Groundwater gradients as they may relate to seepage forces, rates of inflows, feasibility of freezing, etc.

It is usually not practical or advisable to perform field explorations across the entire aquifer. Because of the scale on which groundwater movement occurs, the geohydrologic properties of the various units should be determined at critical and "typical" locations, with the extent of the units being identified by an areawide geologic review. Such a geologic study will reduce the number of required explorations and greatly aid in locating critical geohydrologic boundaries.

In the final analysis, the tunnel will be designed with limited, somewhat conflicting, geohydrologic information. In general, the more care and preparation which are used in developing, executing, and interpreting a geohydrologic exploration program, the fewer the number of unforeseen conditions to be encountered. The following portions of this section detail the major items to be considered in developing an exploration program for tunnel construction.

Normally, field investigations are restricted to within a relatively narrow path along the tunnel alignment, in the order of 100 yards (91.5m) total. If no significant anomalies are encountered along the subsurface profile, i.e., unexpected fracture rock zones or pervious soil strata, this should be sufficient to define the areal groundwater system, when coupled with a regional geologic survey.

When significant anomalies are encountered or the geology of the area is complex, consideration should be given to additional detailed studies. A buried stream channel or faulted zones may represent a substantial recharge source to the site yet appear insignificant when viewed with respect to the total tunnel profile. Sources of regional information are listed in Section 3.21.

3.20 PRELIMINARY INVESTIGATIONS

Geohydrologic investigations are most economical when executed in stages. In the preliminary stage the program should be limited in scope and general in nature. This program will identify the general geohydrologic conditions present, and thus permit identification of the most probable construction technique. After selection of the most likely techniques, preliminary geohydrologic studies can be performed to evaluate subsurface conditions.

3.21 Available Data Sources

In the earliest stages of development, a search for available data should be undertaken. This search should investigate both published and unpublished sources of information. Table 1 is a list of the major sources of available data. Projects in urban areas often cross or parallel major transportation routes. In these cases, abundant data on previously conducted tests and information on as-built conditions are frequently available. In more rural areas, available data are generally less available and of a more indirect nature.

In gathering unpublished data, it is important to know whether indicated locations are the proposed or the as-conducted test sites. A second concern relates to the elevations shown

TABLE 1. SUMMARY OF MAJOR SOURCES OF AVAILABLE GEOTECHNICAL DATA

Published Data

1. U.S.G.S. Surficial Geology Maps
2. U.S.G.S. Bedrock Geology Maps
3. U.S.G.S. Hydrological Atlases
4. U.S.G.S. Basic Data Reports
5. State and County Geologic and Hydrologic maps and reports.
6. National and Local Technical Journals, Magazines and Conference Proceedings.
7. U.S.S.C.S. Soil Maps

Unpublished Data

1. Local test boring and well drilling firms
2. Local and State highway departments
3. Local water departments
4. State Well permit records
5. State and Local transportation departments
6. State and Federal Environmental Agencies
7. State and Federal Mining Agencies
8. Army Corps of Engineers
9. Local consulting, construction and mining companies
10. Geologists, Hydroleogists, and Engineers at local universities
11. Historical records
12. Interviews

Notes: U.S.G.S. - United States Geological Survey
U.S.S.C.S. - United States Soil Conservation Service

on data logs. In many sections of the country several elevation datum planes are used and this simple consideration can lead to significant errors.

Concurrent with the data search should be a reconnaissance of the project area. An experienced observer will study:

- surface topography
- surface water flow and sources
- type and extent of vegetation
- surficial soil conditions
- bedrock outcrops
- existing structures or other physical hindrances to construction operations

Based on the review of available data and site reconnaissance, a generalized assessment of the subsurface conditions can be made. This overview should take into account the effects of groundwater control, not only on the proposed tunnel, but also the surface environment and surface structures. Anomalies should be defined as best as possible and identified for further study during the field investigation.

3.22 Preliminary Field Explorations

After compilation and study of existing hydrologic and geohydrologic information, the preliminary field investigation program can be planned. Typically, the preliminary field exploration program includes 2-1/2 inch (6.4cm) soil borings plus BX or NX size (1-1/8 or 2-1/8 inch diameter) (2.7cm or 5.2cm) rock cores. These borings are generally widely spaced, i.e., 1,000 feet (305m) or more, and are made for the purpose of identifying major subsurface conditions and potential problems. During this phase, the vertical alignment of the tunnels is usually not defined clearly, and therefore the borings are made at easily accessible locations and to depths which are at least two to three tunnel diameters below the most probable invert elevation. With this preliminary information, a more realistic evaluation of the vertical profile is possible. The preliminary explorations should be made by an experienced driller under the supervision of competent geotechnical engineers and geologists. Major significant data can be missed by inexperienced personnel, which can result in serious problems if not discovered until later investigations. The later in the design process a problem is discovered, the more difficult and usually more expensive it is to deal with. Project designs become fixed in a gradual process and many problems discovered late in the process can result in the undoing of significant previous work.

In addition to the detailed logging of the holes by experienced engineering geologists, several additonal routine types of testing should be performed. These include borehole permeability testing of soil, pressure testing of rock, groundwater quality testing, and the installation of observation wells. Individuals observing the explorations should be aware of the impact that an artesian or perched water condition could have on the project and detailed logs of any "unusual" geohydrological conditions should be kept.

3.23 Laboratory Testing

Laboratory testing during the preliminary phase is normally limited to grain size analysis and classification tests. Grain size analyses of a representative number of samples obtained from major soil units are usually sufficient to permit empirical approximations of soil permeability as illustrated in Table 3.

If soft compressible soils are encountered, estimates of compressibility and shear strength parameters must be made. These approximations can be developed through empirical correlations with Atterberg Limit results and standard penetration resistance values respectively, as given in Sections 3.43 and 3.44.

Although water quality is a significant consideration in groundwater control, laboratory testing is usually not included in preliminary investigations. Adequate data to permit a reasonable first approximation of water quality is usually available from the existing information. If no data can be obtained from available sources, a limited number of tests should be performed, primarily to evaluate corrosion and encrustation potentials. The reader is referred to Sections 3.37 and 3.46 for a more thorough discussion.

3.30 DESIGN PHASE FIELD INVESTIGATIONS

The purpose of the design phase hydrologic investigations is to confirm or contest suppositions made in the preliminary data review and exploration program, and to develop parameters for use in final engineering design. The data are used to develop geologic and geohydrologic profiles of the alignment. It involves the collection, assembly and analysis of site-specific measurements from both direct and indirect sources. The program will provide most of the subsurface data to be used for system design and construction evaluation.

The main portion of the field investigation program consists of an exploratory boring program which is usually undertaken in successive phases of increasing detail. Initially widely spaced borings establish the general soil and/or rock profile along the tunnel alignment. Additional borings at closer spacing are then made to search for variations in this profile or to better assess anomalies revealed in the earlier phase. A geophysical

TABLE 2 - TYPICAL RELATIONSHIP BETWEEN SOIL TYPE, GRAIN SIZE AND PERMEABILITY

Coefficient of Permeability, k, cm/sec (log scale)

10^{2} 10^{1} 1.0 10^{-1} 10^{-2} 10^{-3} 10^{-4} 10^{-5} 10^{-6} 10^{-7} 10^{-8} 10^{-9}

<table>
<tr><td>Drainage</td><td colspan="3">Good</td><td>Poor</td><td colspan="2">Practically Impervious</td></tr>
<tr><td>Soil Types</td><td>Clean gravel
d_{10} = 1 mm.</td><td>Clean sands, clean sand and gravel mixtures
d_{10} = 0.08 - 1.0 mm.</td><td colspan="3">Very fine sands, organic and inorganic silts, mixtures of sand, silt and clay, glacial till, stratified clay deposits, etc.
d_{10} = 0.001 - 0.08 mm.</td><td>"Impervious" soils, e.g., homogeneous clays below zone of weathering
d_{10} 0.001 mm.</td></tr>
<tr><td></td><td></td><td colspan="4">NOTE: "Impervious" soils modified by effects of vegetation and weathering.</td><td></td></tr>
</table>

From Soil Mechanics in Engineering Practice, second edition, p. 55, by K. Terzaghi and R. Peck, 1976 and Subsurface Exploration Methods for Soft Ground Rapid Transit Tunnels, Vol. I, by Schmidt, B. et al., April 1976, p. 4-39.

1 cm./sec. = 0.39 in./sec.   1 in. = 25.4 mm.

survey in conjunction with the boring program may be useful in defining conditions between borings and be an aid in selecting additional boring sites. Soil and/or rock and groundwater samples are taken from borings for laboratory testing. Piezometers and observation wells should be installed in completed borings for definition of the groundwater regime (see Section 3.32).

For complete definition of the regional setting, a surface survey is necessary to locate surface water bodies, springs, wells, and other water sources. Estimates of flow rates and seasonal variations need to be made to augment the mapping. Also, infiltration zones and all surface phenomena contributing to groundwater flow should be located and studied. Location of surrounding water and sewer utilities is important in urban settings because, if in poor condition, they may represent sources of seepage.

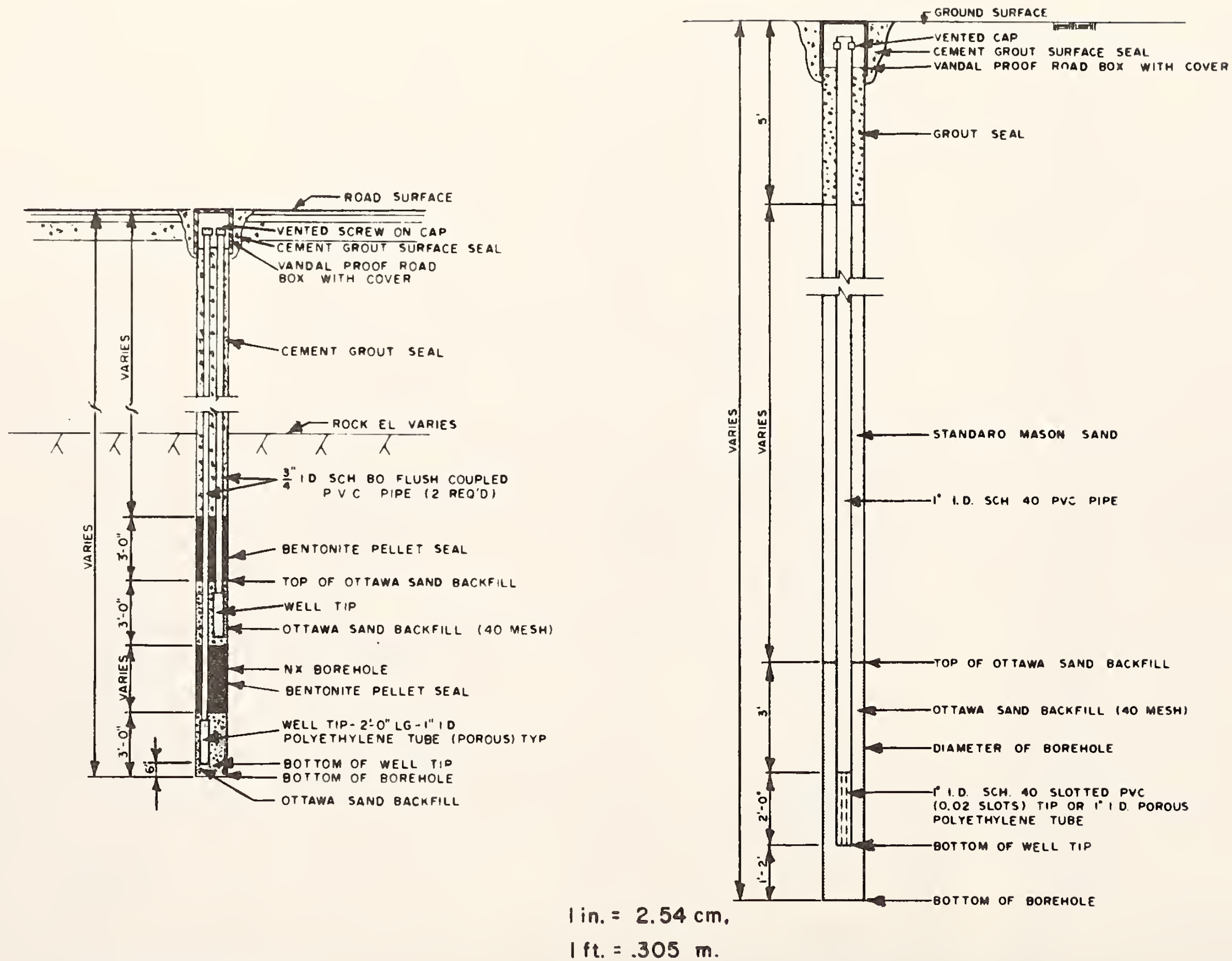
3.31 Test Borings

An experienced drilling contractor should be employed under full time field supervision by experienced geologists to see that borings are classified, labeled properly and, where necessary, delivered for laboratory testing. All samples should be preserved for future examination by the bidders. Soil samples are typically taken at five feet (1.5m) intervals with additional samples taken where strata changes are indicated. In stratified soils below the water table, it is advisable to take continuous samples from a few feet above the arch to a few feet below the invert. There is no general rule of thumb as to the maximum depth of boring below invert, however, a minimum of one tunnel diameter whether in soil or rock is recommended.

The depth where groundwater was first encountered and whether the level subsequently rose (from artesian pressure), receded (indicating perched water) or remained relatively constant should be recorded on all logs. Records should also include indications of soil density and notes as to the increase or decrease in rate of drilling fluid return, problems with caving soils, and relative ease or difficulty in boring advancement.

3.32 Observation Wells and Piezometers

In practice, the term observation well and piezometer are frequently used interchangeably. This is incorrect usage. An observation well is used to measure the water level in an unconfined aquifer. It should not be used in confined aquifers. An observation well is simply a riser pipe with a screened section at the lower end placed in a borehole and backfilled with clean sand and provided with a surface seal. A typical observation well is illustrated in Figure 3.



a) PIEZOMETERS

b) OBSERVATION WELL

FIGURE 3 - Typical Multi-Level Piezometers and Observation Well

A piezometer, while physically similar to an observation well, is sealed at a specific depth. The porous tip of the instrument is isolated from the rest of the borehole by means of a cement grout or clay seal installed around the riser tube or tubes. The piezometer measures water pressure at the tip location and several tips may be installed in a single borehole as illustrated in Figure 3. Water pressure is measured in any one of several different ways including a simple stand pipe, pneumatic pressure transducers, and various electrical transducers.

With observation well and piezometer data, engineers can determine groundwater gradients and flow patterns prior to and during construction. The data are used to evaluate the need for groundwater control, which is a function of many factors, including water pressure at tunnel depth.

Once a method is implemented, observation well and piezometer data are used to evaluate system effectiveness. Since groundwater control methods such as cutoffs, freezing or grouting are designed to maintain groundwater levels outside of the excavation and others such as dewatering are designed to reduce groundwater levels, observation wells and piezometers are essential for proper selection, design and evaluation of a groundwater control system. The number of piezometers or observation wells that should be installed is a matter of judgement depending on the geohydrologic setting. However, as a general rule, it is suggested that groundwater monitoring points be established at intervals no greater than 1,000 feet (305m).

3.33 Geophysical Surveys

Though geophysical surveys have proven useful at times in providing direction to groundwater investigations, they are not always practical, particularly in urban areas. If near surface data are of prime importance, a drilling program may be of most benefit in providing the required information. The decision to use geophysical methods must be made with a good understanding of the capabilities and limitations of geophysical methods in conjunction with a clear definition of the objectives to be determined. In addition, the accuracy and detail required must be well defined, because they have major impact on design and cost of the survey. A further restriction when related to urban area surveys is the problem of data distortion due to traffic, buried utilities, pavements, electrical power, etc.

Numerous geophysical exploratory methods have been applied to groundwater investigations with varying success. The two most commonly used methods applicable to transportation tunnels in urban areas are resistivity and seismic refraction.

Resistivity surveys have been used in locating thick clay layers separating aquifers, delineating aquifer thickness and depth and extent of buried stream channels, defining the fresh-salt water interface and in determining the groundwater table in cases of non-stratified formations.

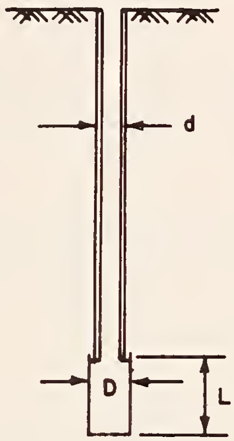
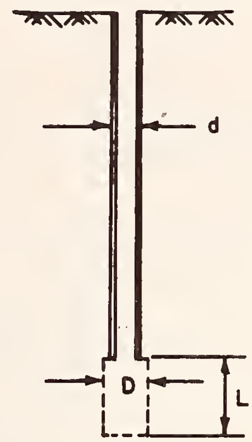
Seismic refraction surveys have also been used in locating buried stream channels and defining aquifer thickness in cases where there is an appreciable velocity contrast between the bedrock and the overlying sediments. Some success has been realized in determining the groundwater table, though if the saturated zone is thin in relation to its depth, the interface may not be detected.

An extensive review of surface and downhole geophysics for use in groundwater investigations has been compiled by the U.S.G.S. including references to specific techniques, applications and case histories. (Refs. 179 and 286). Schmidt et.al. (Ref. 236) also present a concise summary of geophysical methods applicable to groundwater conditions for transportation tunnels.

3.34 Borehole Permeability Testing

Three types of borehole permeability tests are commonly used. They include falling head, rising head, and constant head methods. In general, the falling or rising head methods are used when the permeability is low enough to permit accurate determination of the rate of change of the water level. The flow in the falling head test is from the bottom of the hole into the surrounding soil. Unrealistically low estimates of permeability may be obtained from this type of test due to suspended sediments in the water clogging soil pore spaces. In the rising head test the water level is lowered in the hole and then allowed to recharge with fresh water from the bottom of the hole. If too great a gradient is imposed at the bottom of the hole, there is the possibility of the side walls caving or the bottom heaving. When the permeability is so high as to preclude accurate measurements of the change in water level the constant head method is used. The U.S. Corps of Engineers have developed formulas for determining permeability by the above methods for various soil and water table combinations after Hvorslev (Ref. 139). Permeability calculations for two of the most commonly used borehole tests are presented in Figure 4.

The results of borehole permeability tests indicate, at best, a relative magnitude of soil permeability at the borehole location. The tests tell little about the overall transmissivity of the aquifer and nothing about its boundary conditions. Because of these serious limitations, borehole permeability tests should only be relied upon to identify major differences in permeability

	 CASED HOLE; SOIL FLUSH WITH BOTTOM	 CASED HOLE; UNCASED OR PERFORATED EXTENSION OF LENGTH, L
CONSTANT HEAD TEST	$K_m = \dfrac{q}{2.75 D H_c}$	$K_h = \dfrac{q \ln\left[\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right]}{2 \pi L H_c}$
VARIABLE HEAD TEST	$K_m = \dfrac{\pi d^2}{11 D (t_2 - t_1)} \ln \dfrac{H_1}{H_2}$	$K_h = \dfrac{d^2 \ln\left(\frac{2mL}{D}\right)}{8L(t_2 - t_1)} \ln \dfrac{H_1}{H_2}$

NOTATION:

K_m = MEAN PERMEABILITY $= \sqrt{K_h \cdot K_v}$

K_h = HORIZONTAL PERMEABILITY

K_v = VERTICAL PERMEABILITY

q = FLOW OF WATER

L = LENGTH OF INTAKE ZONE

H_c = CONSTANT PIEZOMETRIC HEAD

H_1, H_2 = PIEZOMETRIC HEAD AT t_1, t_2

d = DIAMETER OF STANDPIPE OR CASING

D = DIAMETER OF INTAKE ZONE

m = $\sqrt{K_h / K_v}$; t = TIME

FIGURE 4 - Typical Field Permeability Test Methods

of subsurface strata. More reliable estimates of permeability can be obtained from grain size analyses and short term (15-30 min.) pumping tests.

Although borehole tests have limitations, they can be done quickly and inexpensively during the boring program. They are useful for preliminary evaluation of in-situ soil permeability.

3.35 Pressure Testing in Rock

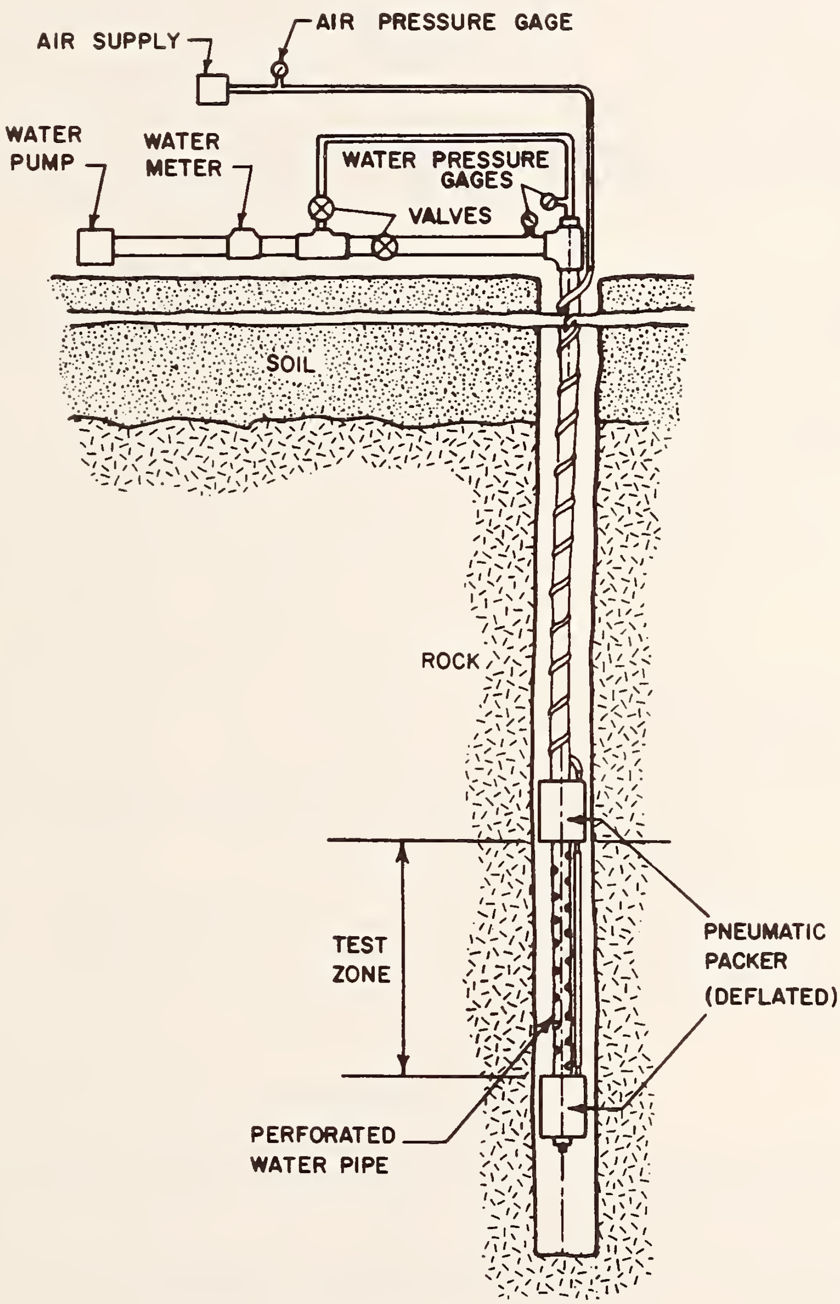
The counterpart to borehole permeability testing in soil is pressure (or packer) testing in rock. In this borehole test, selected sections of the formation are sealed from the remainder of the borehole, into which water under pressure is pumped at a constant rate. The test provides a means of determining the apparent permeability of the rock. Pressure testing is useful in identifying fissured zones in rock and usually provides a permeability value of the correct order of magnitude for practical purposes. A typical test set up is illustrated in Figure 5.

3.36 Limited Objective Pumping Tests

Often exploratory shafts, galleries, and pilot tunnels are advanced for the purpose of preconstruction soil and rock investigations and for tests of construction equipment and techniques. Where feasible, simple short-term pumping tests can be useful in the determination of the type of water source (localized or diffused), and in estimating the order of magnitude of the permeability and transmissivity of the aquifer. Short term tests are more reliable in confined aquifers than in unconfined, unless the latter is specially instrumented, closely supervised and the results interpreted by experienced personnel. This is true due to the significant time required to deplete storage in the unconfined aquifer and thereby reach a steady state flow.

3.37 Groundwater Chemistry

Maintenance costs of a dewatering system can be increased substantially as a result of dissolved elements in the groundwater. Corrosive agents can damage pumps, motors, screens, and piping. Encrusting agents can clog screens and filters, leading to periodic chemical and mechanical cleaning. To select cost effective equipment for the dewatering system, the groundwater chemistry must be determined. These are especially important if the system must run for an extended period of time and/or if it is in a location with a history of groundwater problems. Table 2 summarizes the most common groundwater constituents which affect dewatering systems.



Reprinted from Foundation Engineering Handbook edited by Winterkorn & Fang, by permission of Van Nostrand Reinhold Company

FIGURE 5 - Rock Packer Testing Apparatus

TABLE 3. SUMMARY OF GROUNDWATER CONSTITUENTS AFFECTING DEWATERING SYSTEMS

CONSTITUENT	EFFECT
Calcium Carbonate	--Encrustation of screws and surrounding materials due to precipitation as amount of dissolved carbon dioxide decrease with decreased pressures.
Iron Compounds (>0.5 ppm)	--Encrustation as with calcium --Iron hydroxide deposited as jelly-like substance due to increased velocities. --Black ferrous oxide, red ferric oxide and white ferrous hydroxide form due to increased velocity and exposure to oxygen in dewatered voids. --If water contains less iron than its capable of sever corrosion can result.
Manganese Compounds	--Similar to iron
Bacteria	--Iron bacteria feed on carbon compounds including bi-carbonates and carbon dioxide producing a well clogging slime.
Hydrogen Sulphide	--Corrodes steel and copper based alloys. Copper sulphide encrustation can result.
Carbon Dioxide	--Dissolved carbon dioxide forms carbonic acid which is corrosive.
Oxygen	--Dissolved oxygen accelerates corrosion.
Total Dissolved Solids	--Accelerates corrosion due to increased electrical conductivity.
Hydrogen Ions	--A pH less than 7 results in an acid solution which is corrosive.
Silica	--Will combine with iron and manganese to form encrusting silicates which are insoluble to acid.

Several methods for obtaining water samples for chemical analyses are in present use; bailers being the simplest, but procuring the least representative sample. More sophisticated devices such as gas activated systems (Fig. 6) for collecting samples at selected depths are being used more frequently to obtain high quality samples. In wells of adequate yield, pumped samples may be taken. For optimum results a down hole pump should be used. Suction and air lift pumps are less desirable because they cause the water to de-gas due to the vacuum inherent in these pumping systems thereby modifying the sample.

In all cases, samples should be taken only after the well has been pumped or bailed long enough to assure that all standing water, drilling fluids, and foreign material have been removed from the well, allowing a representative amount of groundwater to enter the well from the aquifer. Even in cases where the well water appears relatively clear at the start of pumping or bailing, stagnant water can contain higher concentrations of iron and corrosion products than is typically present in the aquifer. It is therefore, recommended that two to three times the volume of water stored in the well be removed prior to sampling.

Chemical testing for carbon dioxide, hydrogen sulfide and dissolved oxygen should be performed in the field because concentrations will change in the time it takes to transport samples to the laboratory. The pH is also affected by the gas content, which is another reason field tests should be performed. Water temperature, which affects the corrosion rate and is needed to correct pH to standard conditions, should be measured in the field. Acid is usually added in the field to samples to be tested for iron and manganese to assure that they do not precipitate prior to laboratory testing. Field testing for dissolved iron is also preferable with laboratory testing as confirmation.

3.40 DESIGN PHASE LABORATORY INVESTIGATIONS

Laboratory testing should be performed on both soil and water samples. The testing of soil samples should generally be conducted to establish the gradation of soil units as an aid to evaluation of permeability. Depending on site conditions, it may be necessary to evaluate compressibility and shear strength of soil outside of the excavation limits in order to predict settlement and excavation instability which may occur due to lowered groundwater levels. It is good practice to obtain soil samples at the location of borehole permeability tests. A gradation analysis or visual classification of these samples will aid in evaluating the validity of borehole tests. In highly stratified

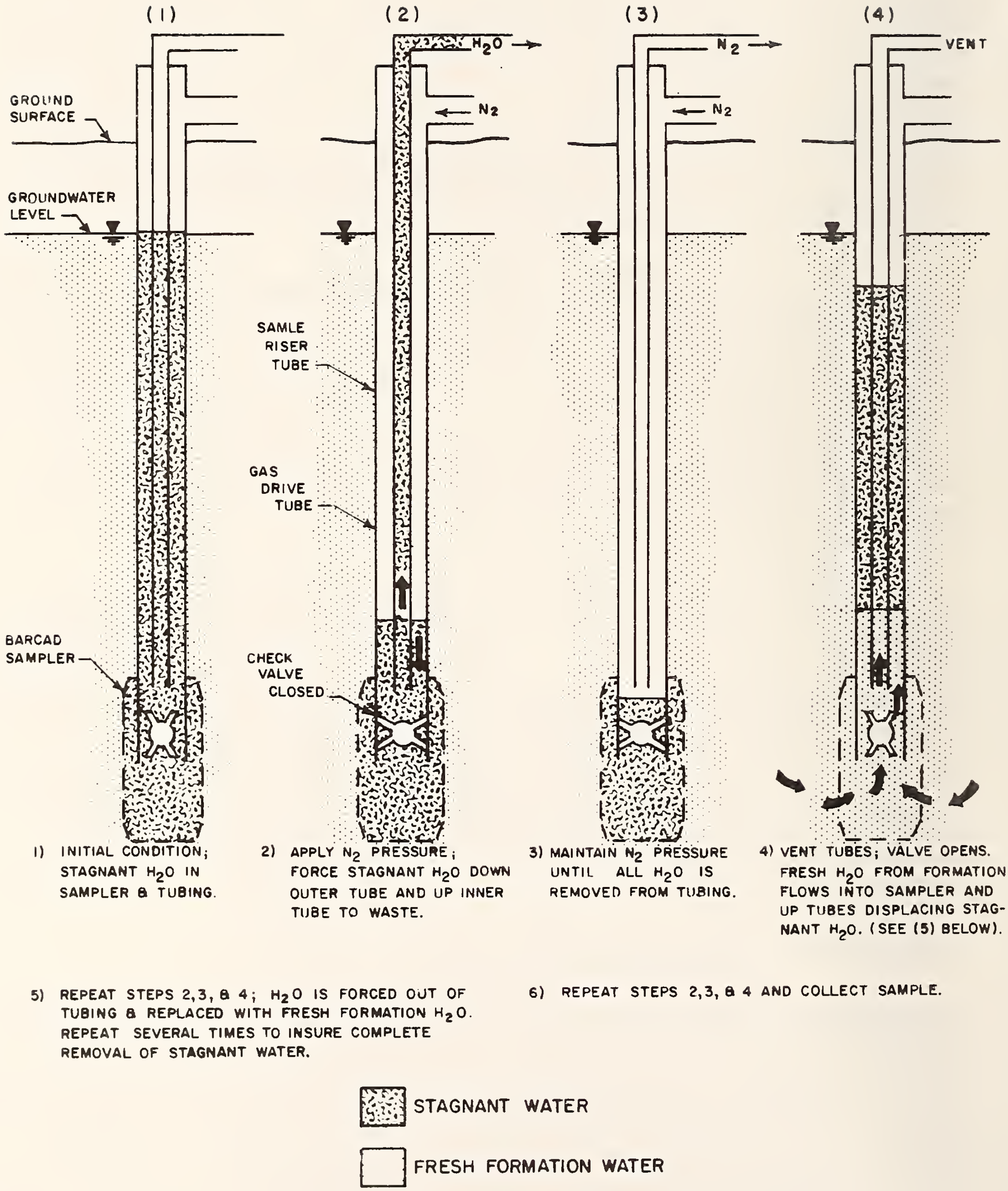


FIGURE 6 - Gas Activated Groundwater Sampler

fine deposits, the collection, logging and photography of small diameter tube samples is useful in evaluating the potential variation between vertical and horizontal permeabiliy.

In general, a large scale laboratory permeability testing program is not warranted. Difficulties in obtaining undisturbed, representative samples make the results of any such program highly suspect, and thus of little practical use.

From a hydrologic standpoint, the laboratory testing of rock cores can, in general, be limited to a detailed logging of cores. Particular attention should be paid to the spacing of fractures with estimates made on the size and condition of the joints. Again, photographs are useful to convey the condition of rock more efficiently than written logs. Specific comments on common tests follow:

3.41 Grain Size Analyses

The results of grain size analyses of samples recovered from borings are useful inexpensive indices of soil permeability. The well-known Hazen formula $k = 100D_{10}^2$ where k is the permeability in units of centimeters/second and D_{10} is the 10%-finer-than grain size in centimeters. Other modifications of this formula are available which use differing coefficients depending on the effective grain size and consideration of porosity and density. This relation was developed for, and is most applicable to, relatively uniform sands. This technique is improved if a sufficient number of representative samples is available thereby permitting application of statistical methods. In certain circumstances, however, the permeabilities indicated may be substantially in error due primarily to soil gradation sampling technique, and in-situ stratification. Typical soil classifications with corresponding D_{10} sizes and coefficient of permeability are tabulated in Table 3.

3.42 Permeability Tests

Laboratory permeameters are occasionally used to provide measures of permeability for remolded coarse-grained soil samples or for undisturbed samples of fine-grained materials. Occasionally, the results obtained are quite useful, such as in selecting filter media, however, more frequently the measurements provide a false sense of accuracy. The weaknesses of the method are built into the testing technique particularly for reconstructed samples. For example, laboratory permeameters are usually set up as one-dimensional linear flow regimes. While this may be satisfactory for remolded coarse-grained materials, it produces values for permeability that are along the vertical axis of any undisturbed sample and is usually perpendicular to the predominant direction of groundwater flow. Soils are typically anisotropic

with respect to permeability and therefore such tests can be totally misleading with respect to in-situ conditions. The orientation of permeameter samples should be selected on the basis of the anticipated direction and magnitude of hydraulic gradients under anticipated field conditions. Other sources of error in laboratory techniques include the chemical characteristics of the permeant, the ability of the sample or its pore fluid to support growth of microorganisms, dissolved or entrained gases, skin effects at the surfaces of the sample, and the great difficulty in reconstructing representative laboratory samples.

3.43 Atterberg Limits

The compressibility and shear strength of fine grained soils can be approximated by examination of the results of Atterberg Limit determinations. The two most commonly calculated limits are the Liquid Limit and the Plastic Limit, although a third one, the Shrinkage Limit, is also defined. Atterberg established a test procedure to determine the moisture content at which soil goes from a liquid to a plastic state (Liquid Limit); a plastic to a semi-solid state (Plastic Limit) and when the soil is no longer saturated (Shrinkage Limit). Terzaghi and Peck (Ref. 287) suggested the following two expressions to relate virgin soil compression to Atterberg limits:

$$Cc = 0.007\ (LL - 10) \qquad \text{Remolded Soil}$$

$$Cc = 0.009\ (LL - 10) \qquad \text{Undisturbed Soil}$$

Where:

Cc = Coefficient of compressibility as defined from a one dimensional consolidation (Virgin Compression)

LL = Liquid Limit in percent

Figure 7a illustrates a relationship between the liquid limit and coefficient of consolidation taken from Department of the Navy Design Manual DM-7, March 1971.

3.44 Shear Strength Tests

If cohesive soils are present, undisturbed samples should be recovered for laboratory shear strength testing. There are many different types of shear strength tests that can be performed ranging from simple unconfined compression tests to anisotropically consolidated stress path shear tests and direct shear tests. A discussion of these various tests is beyond the scope of this report, however the reader is referred to a text by Lambe and Whitman (Ref. 162) for an in depth discussion.

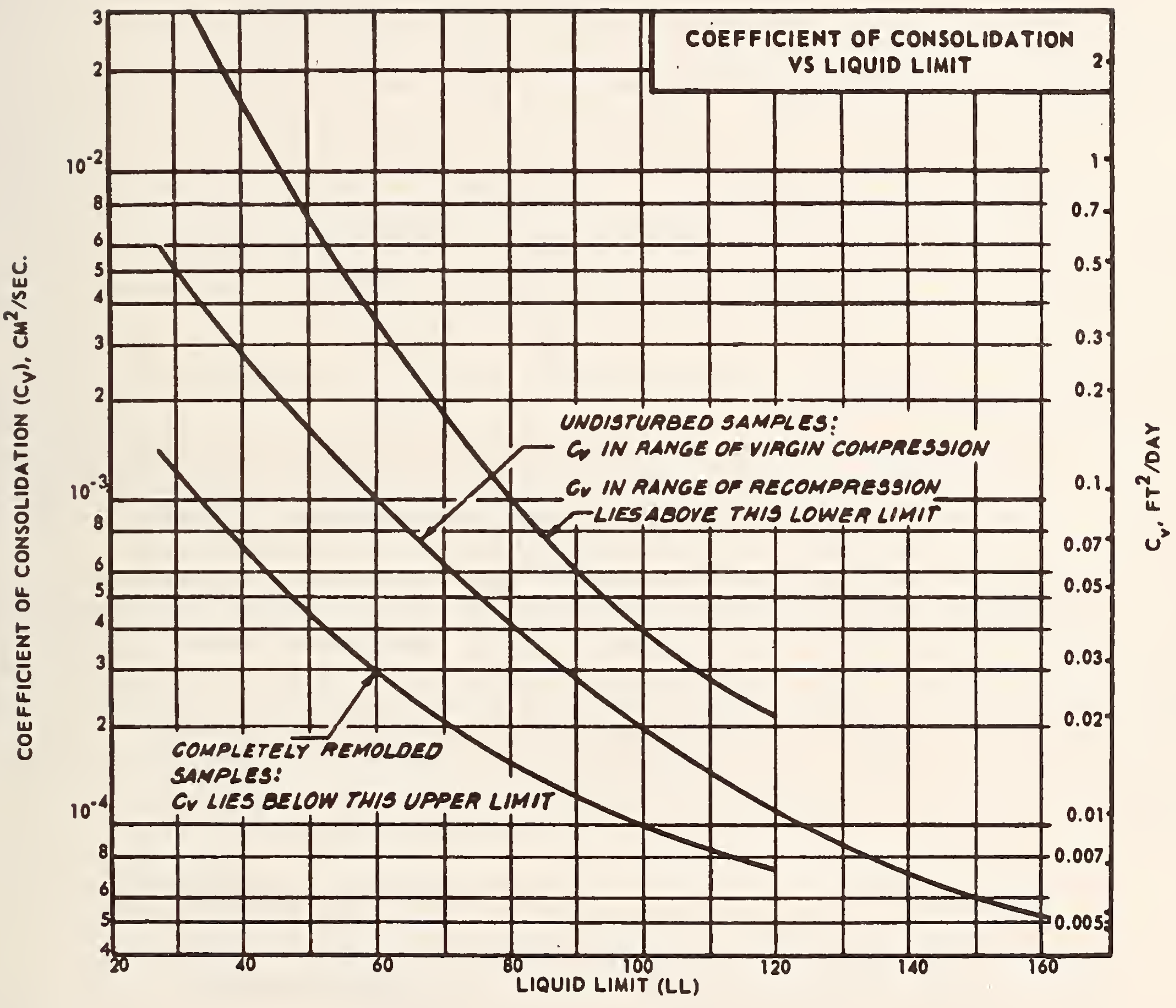


Figure 7a - Coefficient of Consolidation vs. Liquid Limit (Ref. 195)

In addition to providing a rational basis for calculation of material behavior around a tunnel, shear strength information is of significance in determining the possibility of a "blow" either from above or below a tunnel.

3.45 Consolidation Tests

Consolidation tests provide a means for determining of soil compressibility. If soft silt, clays or organic soils are encountered, their compressibility should be investigated. Consolidation testing is one way to do this. The reader is referred to any current soil mechanics text for discussion of consolidation test details.

If the consolidation test results indicate that settlement due to temporary dewatering can result, then a more detailed investigation should be undertaken to evaluate the risks of damage. The alternatives to dewatering are expensive and it will probably be cost effective to carefully compare the costs of underpinning or repair to alternate methods.

3.46 Water Quality Tests

As discussed earlier, an understanding of the chemicals present in the groundwater is required to select equipment and properly anticipate maintenance of pumping systems. Though some confirming chemical testing should be performed in the laboratory, most of the testing should be accomplished in the field. Numerous materials may be found in groundwater, occurring naturally or due to man's influence. To analyze for all such substances would be too costly and time-consuming. Therefore, field or laboratory test requirements should be made with an understanding of which constituents can significantly affect the dewatering system. As a minimum, the following data are needed for a proper evaluation of corrosion and encrustation problems:

- Total Hardness
- Total Iron
- Total Manganese
- Total Alkalinity
- Barcarbonate Alkalinity
- Chlorides
- Sulfates
- Nitrates
- pH
- Temperature
- Hydrogen Sulphide
- Carbon Dioxide
- Dissolved Oxygen
- Total Dissolved Solids
- Silica

Table 2 summarizes the affect of various groundwater constituents on dewatering systems.

When dewatering systems are to be located in the vicinity of industrial, power generation or sewage treatment facilities, additional laboratory analyses should be undertaken to check for pollutants typically associated with these facilities. Industrial pollutants can be quite complex, creating further problems in the protection of the dewatering system against encrustation and corrosion. Petroleum based compounds may cause a hazard from explosion or fire. In addition, polluted water may require treatment prior to final discharge to receiving waters.

The receiving water should also be evaluated for the same substances of concern in the groundwater. When treatment of discharge water is necessary, the condition of the receiving water dictates the degree of treatment required. In cases where treatment is not required, it is good practice to have these data available in case third party claims are made at a later date concerning the effect of the discharge on the receiving water.

Water quality can also have significant effects on chemicals used in grouting or performance of bentonite mixtures used in the construction of cut-off walls. While groundwater conditions will seldom prohibit the use of grouts or bentonite mixtures, they should always be identified so that proper adjustments can be planned in advance of construction.

Groundwater quality considerations are frequently given only cursory attention during design investigations. However, with current environmental laws, groundwater quality considerations can have major impact on project feasibility. The costs of recent subway projects in Boston and Buffalo have been significantly increased due to industrial pollution and naturally occurring dissolved hydrogen sulphide, respectively.

3.50 SPECIAL INVESTIGATIONS

3.51 Pumping Tests

During the primary investigation period, but after the general geohydrological profile has been developed, additional large scale field investigations are frequently undertaken. If it appears that dewatering will be a significant item, the expense of a pumping test will probably be warranted since it is the most reliable means of evaluating aquifer characteristics and demonstrating the response of the geohydrologic regime. The cost of such testing is almost always significant in terms of the cost of the geohydrologic investigation, but may be trivial in terms of the costs of a poor groundwater control scheme. A full scale pumping test can cost $10,000 to $40,000 (1979)

depending on test specifics. The cost of major difficulties in a tunnel under construction due to groundwater problems can easily exceed that amount by an order of magnitude or more.

Pumping test interpretation is not a simple mathematical exercise. Application of the theories involved must be tempered by experience on a case by case basis. The owner often benefits when bidders are given adequate data to form their own individual conclusions.

The scope of the test should be tailored to the complexity of the problem. Tests are performed to determine:

- Transmissivity
- Storativity
- Radius of influence
- Barrier boundary or recharge source
- Time for storage depletion
- Water table surface gradient to be expected
- Effects of surrounding water supplies
- Cost and quality of well construction
- Difficulties of well construction
- Water quality

Detailed design of pumping tests is not presented herein because each test must be site specific, however, a typical test array is illustrated in Figure 7b. A few observations which will enhance the return on the investment are:

- The location and design of the well and piezometers should be based on boring information.

- The well should be of adequate capacity to cause a significant reaction in the aquifer.

- The well should be pumped at a uniform rate for analytical simplicity.

- When recharge sources or barrier boundaries are suspected, piezometer lines should be located such that they radiate in several directions from the well to aid in detecting, location and limits of the boundaries.

- When more than one aquifer is of interest, multiple level piezometers should be installed.

- The test should be continued until the rate of drawdown vs. time is relatively constant. This can be less than 24 hours in a simple confined aquifer and in the order of one to two weeks in an unconfined aquifer.

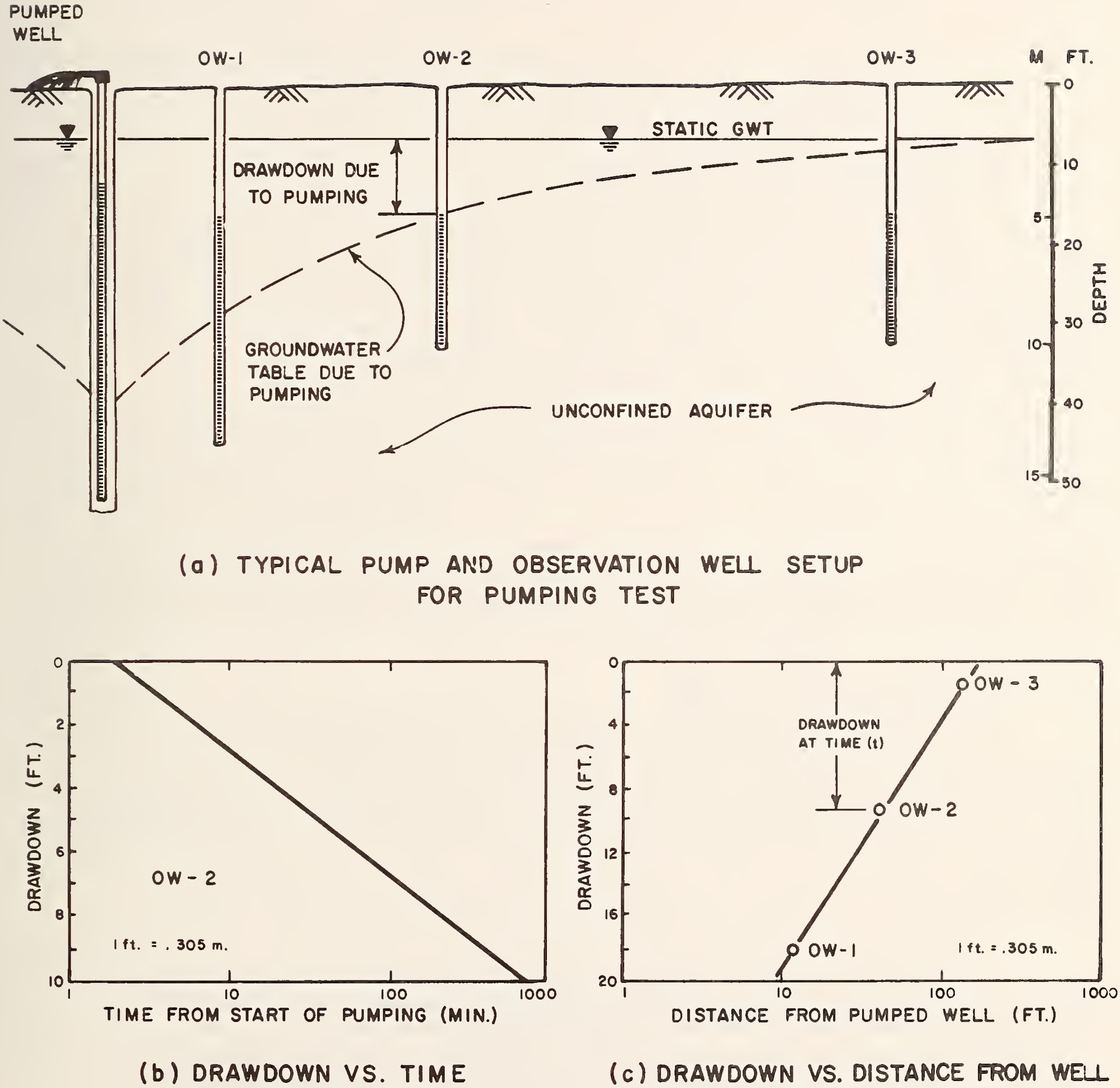


Figure 7b - Typical Pumping Test Array and Data

- Records should be kept of external factors which may confuse test data. These include barometric pressures, rainfall, tidal effects, river elevation changes and pumping rate changes in nearby wells.

- The test should be performed under qualified engineering supervision so that test modifications can be made if necessary.

3.52 Water Treatment Tests

On-site water treatment facilities may be required in rare situations if the groundwater quality is such that it will require special treatment prior to final discharge. Typically the water pumped is of better quality than the receiving water. In rather unique situations, such as areas exposed to unusual industrial wastes, prototype testing of these facilities may be required. In the most typical application, however, the system can be adequately designed based on groundwater and receiving water samples and it's operation then adjusted at the time of installation. A common treatment process is the injection of hydrogen peroxide (H_2O_2) in water containing dissolved hydrogen sulphide (H_2S) which is extremely lethal to aquatic life. It is important to realize that the quality of the discharged groundwater will most probably change as work progresses resulting in a constant need to monitor the system and adjust it for maximum efficiency and minimum cost.

3.53 Environmental Analyses

When standard chemical analyses of groundwater reveal large concentrations of iron, sulphur or manganese ions, it is advisable to test for any iron fixing bacteria. Crenothrix, Gallionella, and other similar bacteria are capable of extracting dissolved ions from their aqueous environment and forming deposits which can clog filters and well-screens. Such bacteria are also suspected of causing corrosive pitting of metal. (See Table 2)

Special efforts may be required to secure adequate samples for the identification of these bacteria because the organism is usually masked by the deposits it produces and is, therefore, not readily noticeable under a microscope. Reducing agents may aid in removing the heaviest deposits allowing the cellular structure of the organism to be observed.

If these bacteria are identified, it indicates that special considerations will be required in selection of pumping system materials and provisions may have to be made for flushing wells and pipes with acid to remove encrustations.

Another biological consideration is the effect of groundwater quality on aquatic life in receiving waters. Suspected toxicity to fish and plant life should be studied by means of special bio-assay testing.

3.54 Special Test Sections

If grouting, freezing, dewatering by electro-osmosis, or difficult cutoff wall construction are planned, then a full scale special test section may be appropriate. The resulting information permits a better technical evaluation of the system, and allows for the development of more comprehensive specifications. A side benefit is an increase in the contractor's confidence in the system. These two factors will likely result in lower construction bid prices with such savings generally more than compensating for the cost of the testing. A full scale slurry wall test was recently completed for the Massachusetts Bay Transportation Authority and grouting tests have been performed in Washington, D.C. and Baltimore in connection with construction of rapid transit systems to demonstrate the effectiveness of chemical and compaction grouting techniques in controlling ground movements around soft ground tunnels.

3.60 ANALYTICAL TECHNIQUES

The ultimate objective of field and laboratory investigations is, of course, design of a groundwater control system. For a water removal system, it must have adequate capacity including a significant contingency, yet it must be cost competitive with other methods. Specific questions to be answered are, what quantity of water is to be pumped, what rate of pumping can be expected from each device, and how much time will be required to achieve the drawdown for a specific system? The answers to these questions will depend on decisions regarding spacing, design and construction of wells, wellpoints and/or pumps and piping of the system.

The field and laboratory investigations provide the designer with soil types and properties, soil profile, approximations of permeability, profiles of phreatic and piezometric surfaces, and location of aquifers, aquicludes and boundaries. Long term pumping tests more accurately indicate the hydraulic properties over a large area, reflecting the effects of multiple aquifer systems, recharge sources and barrier boundaries. The application of the parameters obtained in the field and laboratory to the questions raised above is made through mathematical analyses and simulation techniques. Based on early work by Darcy and Dupuit, formulas have been developed relating the hydraulic characteristics of aquifers to behavior of pumping wells. With these formulas as a basis, recent researchers have greatly expanded analytical capabilities through the use of digital computers.

As is the case in most mathematical modeling, assumptions and generalizations must be made. In the case of groundwater modeling, an aquifer type must be selected reflecting idealized conditions of geometry, soil properties, initial conditions, pumping rate and boundary effects. The formulas will give correct answers for the assumed conditions, however, the validity of the analysis is no better than the assumptions made. Therefore, a considerable amount of experience and judgement is required in selecting the representative aquifer type and related formulas. In addition, an understanding of the sensitivity of solutions to various assumptions must be thorough to permit reasoned judgements as to the validity of calculated solutions.

In some cases the mathematical state-of-the-art has not reached a level compatible to the complexity of the aquifer being studied. If the system is not too complex, and limiting aquifer types can be defined, the solution can be bounded and then approached, again when tempered with judgement gained from past experience. When this approach is not applicable, flow net analyses, performed either graphically or with the aid of electric analog models, can be used to obtain a solution.

Computer application of numerical analysis by finite difference or finite element methods has been found to be of occasional benefit in dewatering design. The complexity of problem formulation and the requirement of large amounts of detailed data is not economically justified for the usual tunnel dewatering applications. Detailed computer modeling lends itself more properly to large-scale regional groundwater studies, rather than the temporary localized effects associated with tunneling projects.

The data compiled during a proper field and laboratory investigation represents only a minor fraction of the volume of subsurface material involved. The design process, therefore, becomes an exercise in engineering judgement with minimal data being utilized within a manageable mathematical framework. This approach coupled with proper construction supervision and performance monitoring has shown to be the most acceptable from an engineering and economic viewpoint.

With regard to groundwater control methods other than dewatering or freezing, the applicable analytical methods are highly empirical. Preconstruction evaluation of grouting schemes is almost totally dependent on a judgement relative to soil or rock permeability plus simple volumetric relationships upon which estimates of grout volume can be made. Design of cut-off systems is also an empirical process based on soil and rock classifications although seepage studies, using flow nets drawn either manually or by computer, are usually performed to arrive at a proper design of wall depth.

Freezing systems are also often designed from empirical rules based on previous experience. However, recent studies of permafrost conditions accompanying the development of arctic regions have advanced our knowledge of the thermal and structural behavior of frozen soils to the point where quite sophisticated design procedures are available. The value of these may be limited by the lack of basic data on the characteristics of the thermal and structural properties of the soils to be frozen. Special laboratory testing of soil specimens, particularly in the frozen state, will provide these data, and lead to a better technical evaluation of the freezing system in the design stage.

Design of compressed air and slurry or earth pressure balance shields is based on relatively simple relationships which consider tunnel depth and hydrostatic pressure. Work has been done on prediction of face stability as a function of soil shear strength, but final decisions are usually based on previous case information.

3.70 CONSTRUCTION OBSERVATION AND REEVALUATION

Dewatering is a method which relies primarily on advance subsurface information for design. The number, spacing, and capacity of the drainage devices are selected based on the aquifer parameters which have been estimated during the investigation program. During construction, interest in subsurface conditions must intensify. Detailed logs of boreholes made during construction can be an invaluable aid for identifying problem areas which may not have been noted prior to construction. This information is sometimes not provided. A feedback loop of information should be developed within the project management system so that the groundwater control system can be modified as it is installed. Substantial variations in soil conditions may occur, and a groundwater control system which has been designed on the basis of limited data may not be adequate for the entire job. The spacing and capacity of drainage devices or extent of water exclusion systems will probably vary along the tunnel alignment. Slight gradational changes can result in dramatic changes of soil permeability, which can in turn require closer spacing of lower capacity wells (or vice versa), deeper cutoff walls, etc.

Experienced contractors test the performance of their system as it is installed to insure that it is capable of controlling the groundwater as desired. For instance, if a dewatering system has been designed as deep wells on 1,000-foot (305m) centers but variations in subsurface conditions dictate a well spacing of 700-foot (213m) centers to obtain the required drawdown, the only option left open may be to install additional wells as necessary on 500-foot (152m) centers. If the system had been tested as it was installed, this situation would have become apparent much sooner and the initial spacing could have been

decreased accordingly. In the same way, if only 1,000-foot (305m) centers were really required and wells were installed on 700-foot (213m) centers, more wells than necessary have been constructed and the money wasted.

Similar monitoring and adjustment is also required with other groundwater control methods. The cost of a groundwater system may be compared to an insurance policy. If guaranteed results are desired, an excess premium will have to be paid. If a lower premium is paid, the contractor assumes increased risk. A changed condition clause may be employed to attract bids with lower contingencies. The assignment of risk among the Contractor, Engineer, and Owner is a matter of economics, equity and the competitive realities of bidding. A discussion of risk allocation is included in Section 6.30.

Flexibility during design and construction of groundwater control systems is necessary to achieve desired results. Identification of problems ahead of time can save costly delays. Monitoring programs are frequently included to detect problems before they become severe. A major element in any monitoring program is piezometer installations. Peizometers may be used to measure aquifer response to pumping; they may be used to measure the performance of the system; they may be used to demonstrate performance to a specification, as well as to provide additional information on the subsurface conditions.

Piezometers and observation wells may be used to indicate problems with dewatering system operation. Installations which show an initial drop in groundwater level and then begin to rise may be indicating decreased system efficiency. Likewise, measuring the operating level in a well can be an indication of whether or not encrustation is occurring or whether or not construction procedures are adequate to produce minimum well loss.

Monitoring the quality of discharge water can be significant in analyzing a problem which is occurring gradually. In a situation such as tunneling, where wells are turned on progressivly as a tunnel advances, the possibility of encountering water of different quality can occur. An example of this occurred in a cut-and-cover tunnel in Jamaica, New York. Deep wells were being used to dewater the tunnel. This same water was to be injected into a recharge well system at a distance from the site. As the excavation progressed, wells behind the excavation were turned off and those in advance were operated. In one case, turning on two wells in advance of the excavation produced water of significantly higher dissolved iron content. This increase in iron resulted in clogging of the recharge system, thereby requiring an extensive maintenance program to remove encrusted iron.

Although piezometers are the primary tool for monitoring groundwater control systems, many other instruments are used. Table 4 is a tabular summary of some of the more common instruments.

Monitoring can be very helpful in avoiding unforeseen difficuties and in providing data for settlement of future claims. Subsurface investigations do not end when the contract is awarded to the lowest bidder. Data that will be helpful to all parties concerned with the project should be developed continuously in order to provide the most economical and flexible groundwater control system capable of providing the required results.

3.80 SUMMARY

Subsurface exploration programs undertaken for design of tunnels frequently consider groundwater as a factor of secondary interest. Yet it is generally recognized that improper control of groundwater is probably the greatest hazard faced by tunnelers.

Exploration programs are normally undertaken in a minimum of three phases. A preliminary phase which includes collection of all available data plus a boring program of relatively few widely spaced borings. Piezometers should be installed in every preliminary boring. Basic project feasibility and first cost estimates are normally based on the preliminary information.

The second phase is the project design phase which is frequently undertaken in several stages of progressively increasing detail. The major source of information is a well documented test boring program. Experienced field supervision and description of this program is essential. Again, piezometers and/or observation wells should be installed in a representative number of the completed borings for proper definition of the groundwater regime.

Geophysical surveys may be performed to supplement boring information, but they are of secondary importance to boring and piezometric data.

Borehole permeability testing is useful to locate strata with major differences in permeability, but there are many major factors which render these data questionable. Of greater value in evaluation of field permeability determination is proper sample classification and pumping tests.

Packer tests in rock are similar to borehole permeability tests in that they provide data on major differences in strata permeability. Borehole test data, whether in soil or rock should not be relied upon without other confirmatory laboratory and field information.

TABLE 4. SUMMARY OF GROUNDWATER CONTROL MONITORING INSTRUMENTS

Instrument	Use
1. Piezometers and Observation Wells	- Measurement of piezometric levels associated with all groundwater control methods. (See Figure 3)
2. Surficial Settlement Measurement	- Measurement of surface settlement resulting from groundwater control. Usually monitored by optical survey, but can be done with water level devices and pneumatic settlement systems.
3. Deep Settlement Devices	- Measurement of settlement at depth. Can be a single deep anchor device (Figure 8) or a multi-point system (Figure 9).
4. Lateral Movement Devices	- Measurement of lateral movement adjacent to open cuts and bored tunnels in soil and rock (Figure 10).
5. Thermocouples/ Thermistors	- Measurement of temperature in the ground for evaluation of freezing processes (Figure 30).
6. Accoustic Emission Monitors	- Measurement of grout travel in soil and rock. A method still in research.
7. Resistivity	Measurement of chemical grout travel in soil. A method still in research.

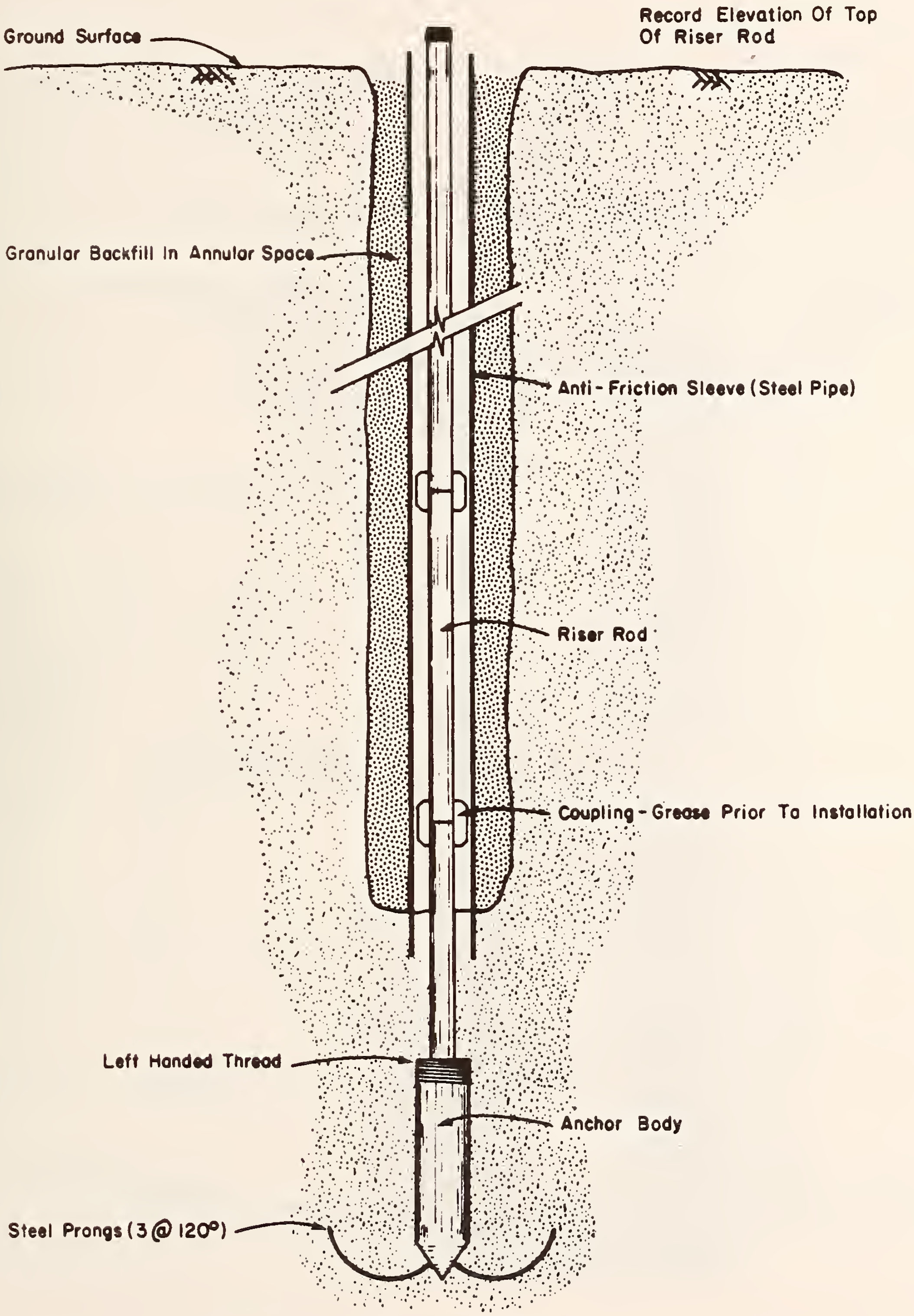


FIGURE 8 - Schematic of a Deep Settlement Monitoring Anchor

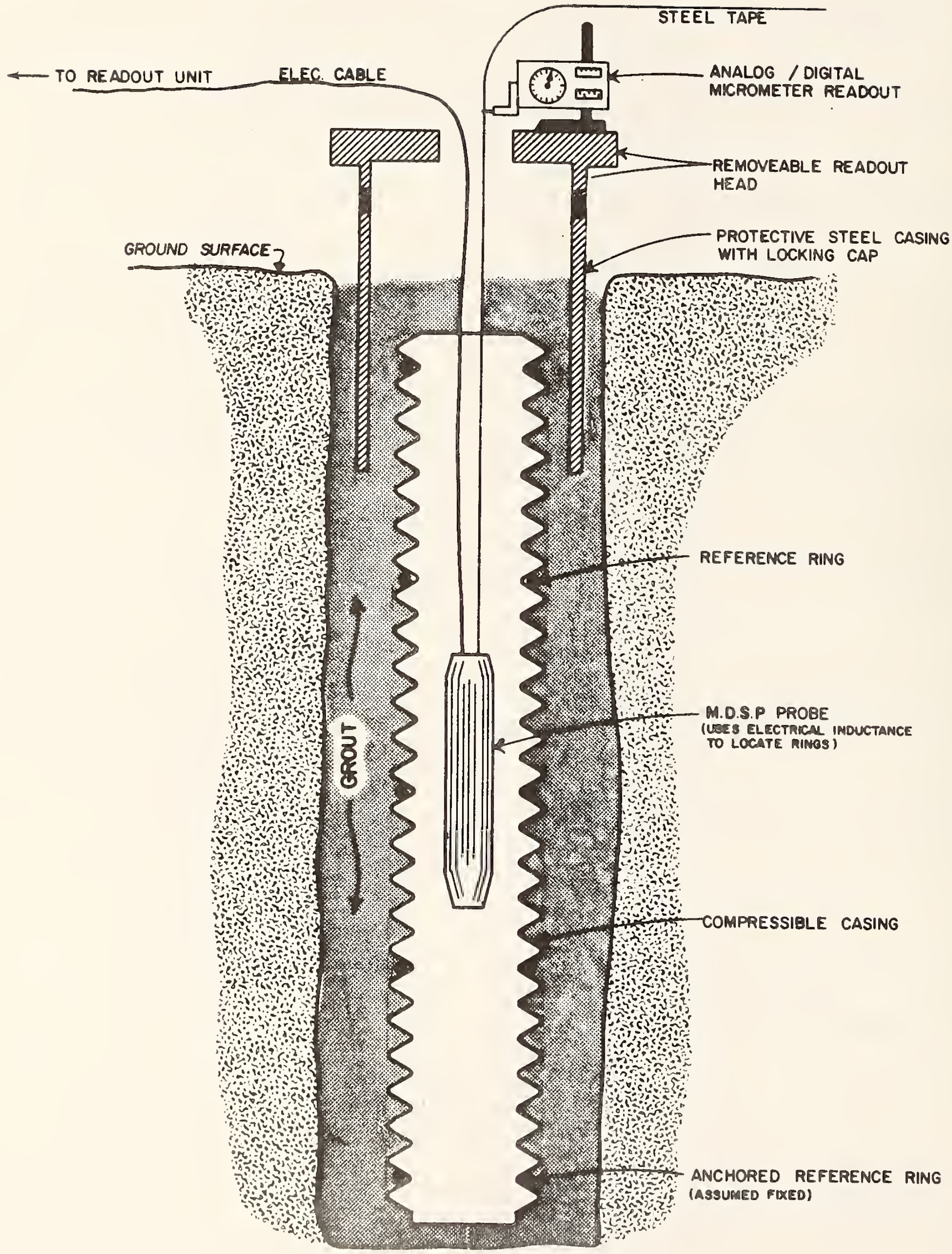


FIGURE 9 - Schematics of a Multi-Point Deep Settlement Monitoring System

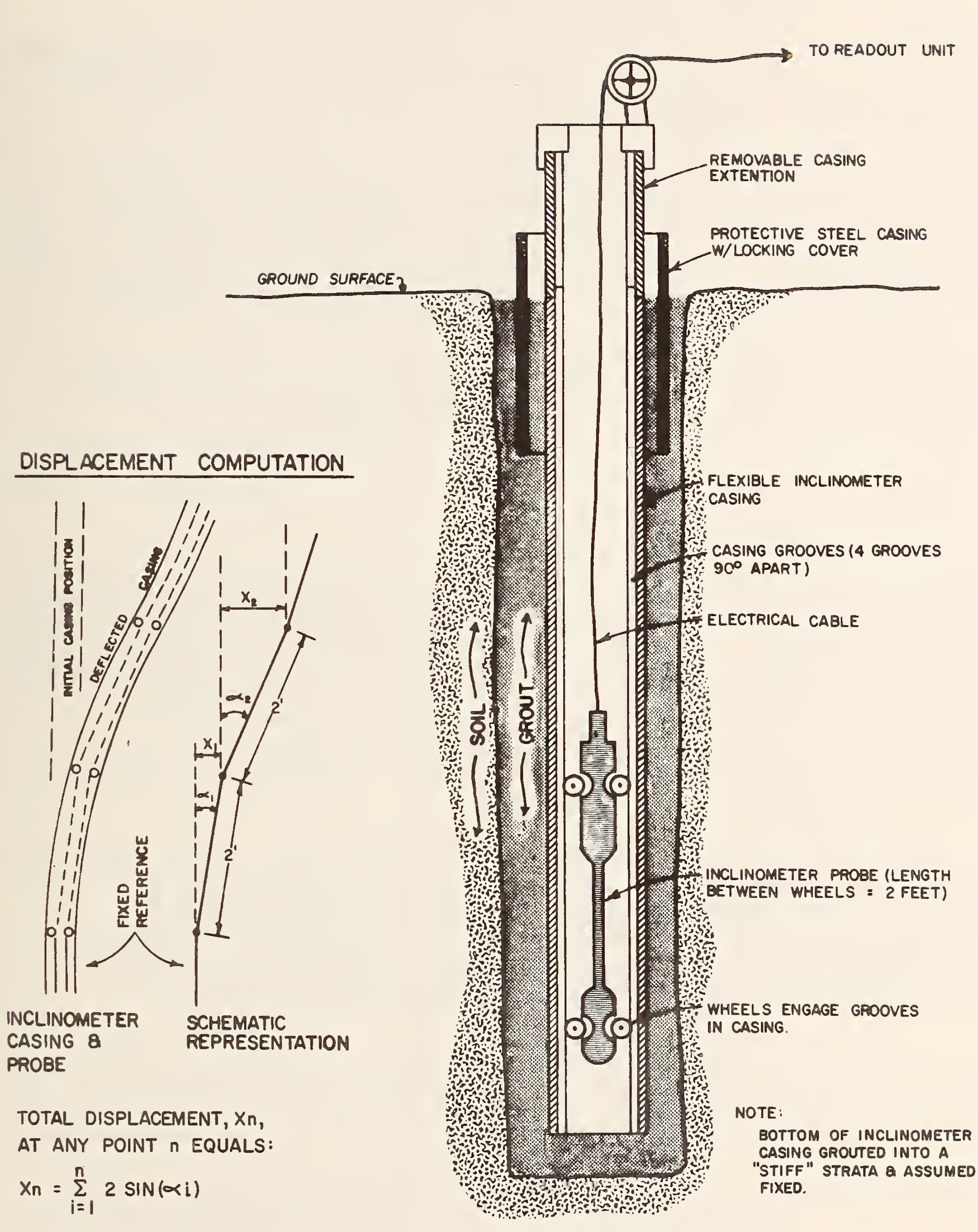


FIGURE 10 - Schematic of Inclinometer System

In addition to the investigation of the hydraulic nature of an aquifer it is important that groundwater quality be determined. Water quality has important influences on selection of equipment, i.e., corrosion and encrustation problems, and on the environmental concerns about water disposal.

For evaluation of aquifer permeability, probably the best laboratory tests are simple grain size analyses. Other tests may be necessary if soft compressible soils are encountered. Such tests include Atterberg Limit determinations, compressibility, and shear strength tests to help in the evaluation of possible effects of dewatering such as settlement and lateral movement.

One of the stages of design phase investigations is special investigations which are normally made following thorough study of all conventional data, i.e., test borings and laboratory data. Special investigation may include long term pumping tests, full scale test sections, and extensive environmental analyses such as bio-assay tests to determine tolerance of plant and aquatic life to proposed methods.

Analytical methods for all groundwater control methods are a combination of rigorous solutions to physical problems and empirical methods. Computer methods have greatly expanded our ability to evaluate groundwater behavior, but the complexity of problem formulation and the large amount of detailed data required usually is too great for economic justification for the solution of "normal" groundwater control problem. Most system designs are an exercise in judgement within a manageable mathematical framework.

The third phase of any subsurface investigation is construction observation and reevaluation. Subsurface investigations do not end when the contract is awarded. Data are continually developed during the installation and monitoring process which must be reviewed and processed through a feedback loop to permit system modification as necessary to achieve desired results.

4.00 GROUNDWATER CONTROL METHODS

4.10 GENERAL

Groundwater control can be accomplished either by extraction of groundwater during or in advance of construction, or by exclusion of the water from the work area during construction to prevent seepage induced problems. Eight groundwater control methods are considered. Case history information is provided as appropriate to illustrate the uses of the methods as well as their limitations. Comparisons are not made; however, criteria for the selection of appropriate groundwater control methods are discussed. These criteria include technical, cost, and risk considerations. A careful study of the advantages, disadvantages, and limitations of each method is necessary for selection of the best approach to control a specific groundwater condition. Selection criteria are discussed in Section 5.00.

4.20 DEWATERING

Dewatering is accomplished by one, or a combination of several techniques, which remove groundwater by pumping to provide "workable conditions" in the tunnel. "Workable conditions" can range from perfectly dry and stable to relatively wet, depending on soil type, head of water, and tunneling technique. Dewatering is typically used for one or more of the following reasons:

1. To lower the water below tunnel invert.
2. To lower the water level as close as possible to an impervious layer.
3. To lower the water level so that compressed air needs can be reduced.
4. To reduce water pressure in an underlying confined aquifer close to, but not necessarily penetrated by the tunnel.

Dewatering can be accomplished either in advance of tunneling by predrainage or by pumping within the tunnel. It is not unusual to combine predrainage and internal pumping to adequately control groundwater.

The three most common predrainage methods are deep wells, ejectors, and wellpoints.

The basic elements of each method are illustrated in Figures 11 through 13, and each method is discussed in the following sections.

4.21 Deep Well Systems

Deep well systems employ individual pumps located at each drainage location. A pump is installed inside each well and is powered either by a submersible or surface mounted electric motor. A significant advantage of well systems is that large quantities of water can be controlled from relatively widely spaced pump locations. The spacing depends upon aquifer characteristics. In uniform coarse-grained aquifers, the required flow rates may be on the order of thousands of gallons per minute, while in stratified, well-graded, or fine-grained soils, flows on the order of tens of gallons per minute may be sufficient to adequately control groundwater.

The ideal well should be constructed to a depth which is sufficiently below the bottom of the tunnel to enable the water level "crown" which develops between adjacent wells to be below the bottom of the tunnel. The concept of a crown is illustrated in Figure 12 for a wellpoint installation. The "crown" between drainage devices controls the extent of the system required. The more permeable the material, the flatter the "crown", and vice versa. Therefore, wells in more permeable material can be spaced wider than those in less permeable material.

Dewatering at the interface between an upper permeable and lower impermeable layer is difficult because the water level can never be lowered below the top of the impervious layer, except perhaps at pumping locations. Because wells cannot completely drain the upper material, seepage induced runs of soil into the tunnel may occur. This perched water must be handled either by very close spacing of wells, by other predrainage devices, or by some other method such as grouting, pumping from the face, freezing, or breasting the face with lagging or special plates.

Since there is a pump at each deep well and because wells are usually drilled to some significant depth, a large investment is involved for each installation. It is therefore prudent to carefully design the well to maximize capacity, ensure a reasonable lifespan, and minimize maintenance requirements.

Wells have been used in many tunneling projects in recent years, such as for a subway extension in New York City, which was successfully dewatered with wells as described below. The aquifer was a uniform outwash sand extending about 60 feet (18.3m) below subgrade. Most sections were constructed as "cut-and-cover," except where crossing under major highways. Cut-and-cover construction allows considerable flexibility with respect to dewatering,

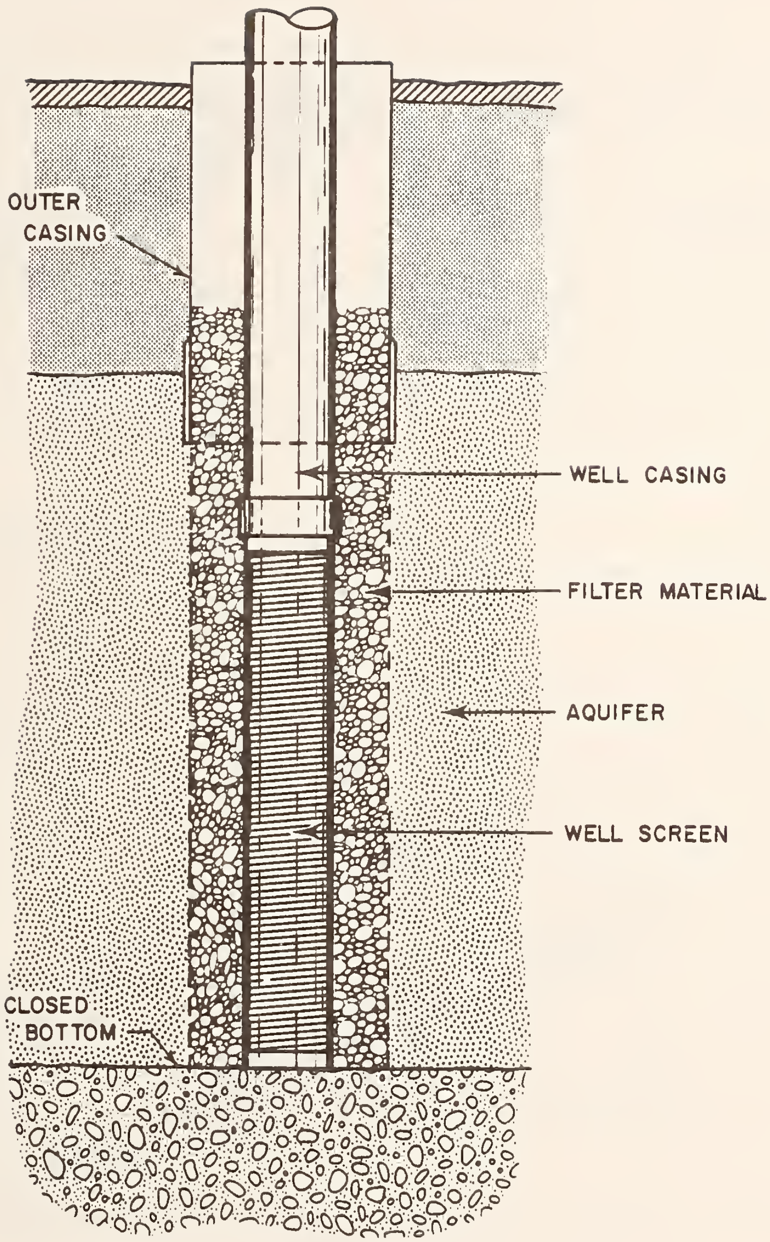


FIGURE 11 - Typical Deep Well

such that control of minor residual water is greatly facilitated. A disadvantage in New York City is that union work rules require one operating engineer for a maximum of five deep well pumps. Prudent scheduling and operation of the well system is therefore required to minimize the number of pump operators.

Where the spacing of drainage devices is relatively close, i.e., less than 50 feet (15.2m), due to low permeability or to the necessity to dewater as closely as possible to an impermeable layer, it is generally advisable to use ejectors or wellpoints.

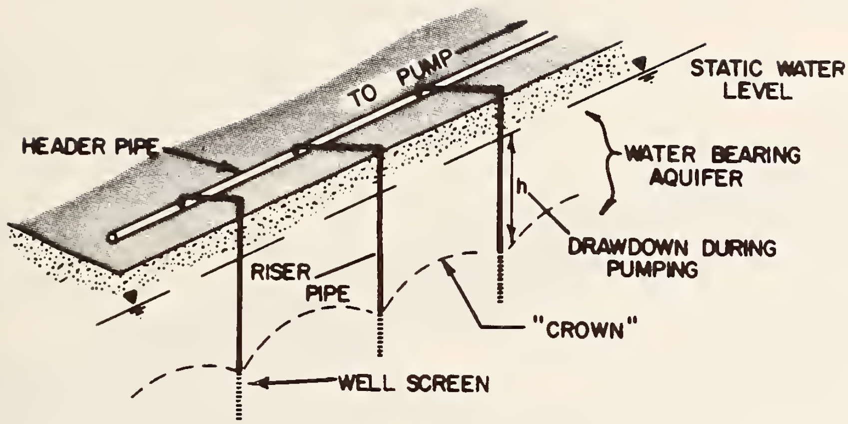
4.22 Wellpoints

Wellpoints are connected to a common suction header which is evacuated by a pump. (Fig. 12a) The system depends on atmospheric pressure to lift the water and, hence, has a practical limit of 15 feet (4.6m) for lowering water levels. With special procedures and equipment, this can be increased to 20 feet (6.1m). If greater drawdown is required, multiple stages will be necessary, as illustrated in Figure 12b.

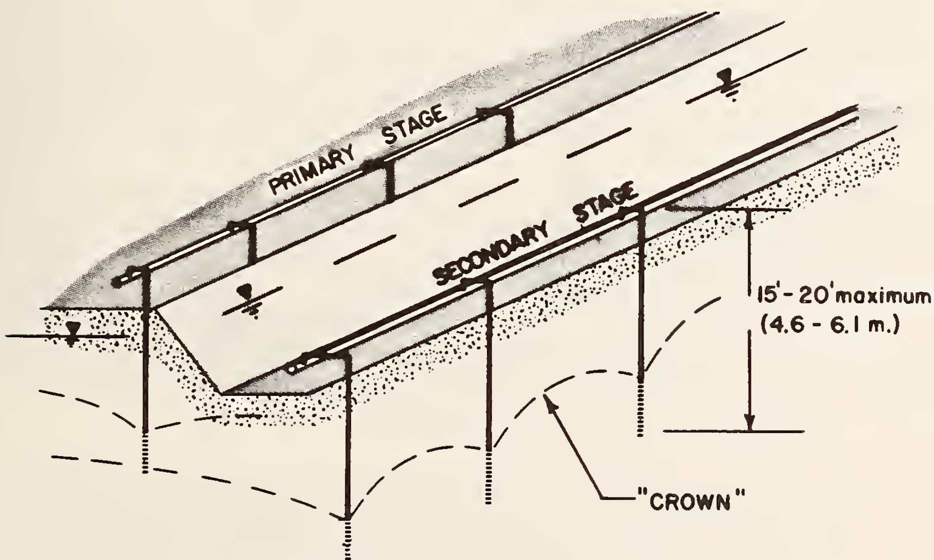
Because of the lift limitation, wellpoints are usually impractical for bored tunnels, except very shallow bores, or for occasional remedial measures at the face.

Wellpoints are sometimes used in cut-and-cover tunnels and in shafts. Because of the low cost per unit, they are effective in stratified soils where close spacing is required. Wellpoints may be less preferable than other predrainage methods because of delays in installation of successive stages; the interference of the wellpoint equipment with excavation, ground support and other operations; and difficulties in coordinating excavation backfilling operations. Modern developments in equipment, including vertical wellpoint pumps, have reduced some of the objections. Deep wells or ejectors, which can be placed outside the excavation, are frequently used in preference to wellpoints. Although their cost may be higher, the difference is usually more than compensated for by the savings in other operations. In most urban areas, union work rules produce an economic disadvantage in the use of wellpoint systems as compared to technically feasible alternatives such as deep wells.

A sewer tunnel in Richmond, Indiana, illustrates a situation where deep wells proved fruitless in a water table aquifer underlain by glacial till, and wellpoints were used in both shafts and at tunnel invert to successfully dewater the project. The till was located only a few feet below tunnel invert and acted as an impermeable layer overlain by a shallow aquifer. The aquifer consisted of a loose sand and gravel deposit. Unusually heavy rains at the time of construction and a resultant rapid infiltration of rainwater to the water table resulted in a 10-foot (3m)



a) SINGLE STAGE



b) TWO STAGE

FIGURE 12 - Typical Wellpoint Installations

rise in water level above tunnel invert. Preconstruction explorations indicated only 1 foot (0.3m) of water above invert, and original plans were based on the assumption that sumps in the tunnel would be adequate. The deep wells were incapable of lowering the water to invert level, and the use of wellpoints proved successful in controlling the problem.

Wellpoint systems are relatively inexpensive to install and are flexible in that adjustments to well locations and spacing are easily made provided that header and pump sizes have been conservatively designed. By using a vertical casing with vacuum and a well pump, successive header stages can be tied in to achieve deeper dewatering, while the pump station can remain at the surface, out of the excavation. The use of additional stages, however, usually increases the price of the wellpoint system to a point where ejectors are more economical.

4.23 Ejectors

The ejector uses a water driven jet pump (nozzle-venturi combination) to remove water from the device (Fig. 13). Water is supplied to the jet pump under pressure by a high-head centrifugal pump attached to a storage tank. The discharge of the centrifugal pump is connected to a distribution header which supplies the ejector jet pumps. The combined flow of supply water plus groundwater is pushed up the riser pipe and enters a collector manifold and flows into the storage tank. The excess groundwater is then overflowed from the storage tank or pumped to discharge. Ejectors require the installation of two pipes into a borehole, the supply line and return line. Sometimes these pipes are concentric with a short wellpoint screen at the bottom, and sometimes they are constructed as two separate pipes which can be installed into a larger diameter borehole with well screen and casing.

Ejector systems are intermediate between wellpoints and wells. They are capable of removing water at depth without the use of multiple stages, and, like wellpoints, they can be more economically spaced at small intervals to dewater close to the top of an impermeable layer. Ejector systems are highly susceptible to iron or manganese encrustation. If the water contains high concentrations of dissolved iron or manganese, continuous maintenance can become costly.

Ejectors rarely exceed an efficency of 25%, so the required horsepower is higher by a factor of 4 or 5 than for wells or wellpoints at the same head. In addition, economic considerations usually limit the use of ejectors to systems with low yield per unit on the order of 5 to 10 gpm (0.3 to 0.6 l/s) per ejector. Since ejectors pump air as well as water, they are capable of

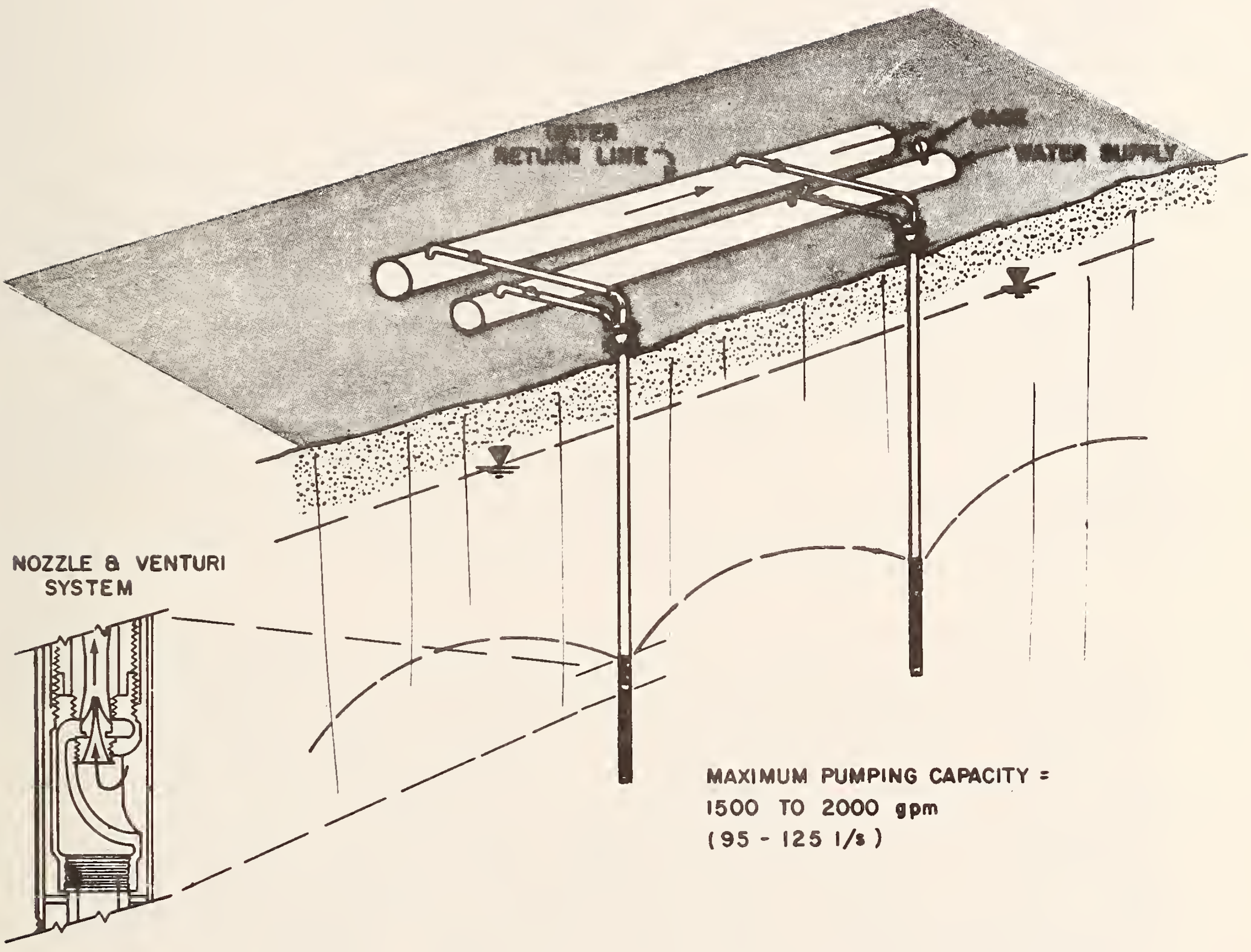


FIGURE 13 - Typical Ejector System

creating a vacuum in the surrounding soils when the inflow of water is very low. This can have a dramatic effect in stabilizing fine sands and silts.

One example of a difficult dewatering problem was encountered on a section of a subway in Osaka, Japan. The 32.8-foot (10m) diameter tunnel was driven using a wheel-type excavator under compressed air. The tunnel machine was driven into an existing station excavation, and had to be dismantled and reassembled to drive off at the other end of the station. In order to set the shield, lock, and muck handling apparatus, approximately 200 feet (70m) of tunnel had to be driven before the air could be turned on. The lower third of the face was in a stiff clay. Directly above the clay was a 10-foot (3m) layer of a silty sand and above the silty sand was clean sand and gravel. Thus, the stratification was coarse sand, over fine sand, over clay--a difficult sequence to predrain completely. A system of wells had originally been provided for dewatering the 200-foot (70m) free air length of tunnel, but they were only partially effective. The volume of excavated soil far exceeded the tunnel volume until a collapse to the surface occurred, stopping the job. A system of ejectors was installed on both sides of the tunnel for the length of the free air section, and once the system was in operation, the face stabilized, and the contractor was able to set his air locks and begin compressed air tunneling without difficulty.

4.24 Pumping Within the Tunnel

Pumping water as it enters a tunnel is a common means of dewatering. In material which cannot be predrained effectively, such as rock, glacial till, and clay, it can be the main drainage technique. In such materials, the tunnel, with a permeable liner such as ribs and lagging, acts as a large horizontal drain and is capable of intercepting more water throughout its length than any other predrainage device. If the material encountered is stable in the presence of water, then the cheapest and most effective way to handle the water is to remove it after it enters the tunnel. Usually, pumping within the tunnel is necessary to supplement other predrainage techniques and also as a matter of housekeeping.

The extent of the system can vary from modest sumping at the face to elaborate undercuts and drifts which can form a drainage maze throughout the length of the tunnel. A section of the Washington D.C. Subway was driven in free air with minor sumping from the tunnel face. Wells were used around shaft

areas and also in combination with grouting to stabilize critical areas. The remainder of the alignment was in clay and the minor seepage from occasional sand seams and pockets was readily handled at the tunnel face.

In contrast, a section of the San-yo Line, part of Japan's National Railways, required a complex system of drainage tunnels and drifts. The tunnel had to be cut through badly faulted granite, and flows of thousands of gallons per minute were handled by the drainage system. To advance the main tunnel 0.25 miles (0.4km) the contractor developed 0.75 miles (1.2km) of branch tunnels and 5 miles (8km) of 5-inch (12.7cm) diameter drain holes.

When choosing a dewatering method for groundwater control in tunneling, a commitment must be made well in advance. The progress of the work is critical, and if an unforeseen water problem arises, the cost of handling it can become very high. Stopping or impeding the excavation progress leaves no place for the heading crews to work. Predrainage systems must be able to deplete stored water, which can take a long time, especially in fine-grained soils. If the decision is made to handle water from within the tunnel, the method of tunneling and type of tunnel machine should be designed to be capable of handling water.

Dewatering can cause detrimental effects such as settlement or exposure of timber foundations. If these effects are serious enough, it may be necessary to develop a recharge scheme.

4.30 RECHARGE

Recharge is the replenishment of groundwater outside the construction area in order to prevent detrimental effects of lowering groundwater levels. The reasons for using recharge vary, but usually they are to protect existing structures or the environment. Lowering the water level can cause settlements in structures underlain by compressible soils as illustrated in Figure 14. Exposure of timber foundations can cause rot. Removal of water from the roots of plants can kill them. Water levels in ponds and streams may drop when surrounding groundwater levels are lowered. A depletion of the groundwater resource can also limit water availability in areas where the water supply is drawn from the same aquifer being pumped for construction.

Recharge is applicable in granular soils of the same general gradation as those which can be successfully dewatered. However, because of encrustation problems, attempts at recharge by injection are usually limited to soils no finer than medium sand. Recharge by infiltration can be successful in finer soils such as fine sands and coarse silts.

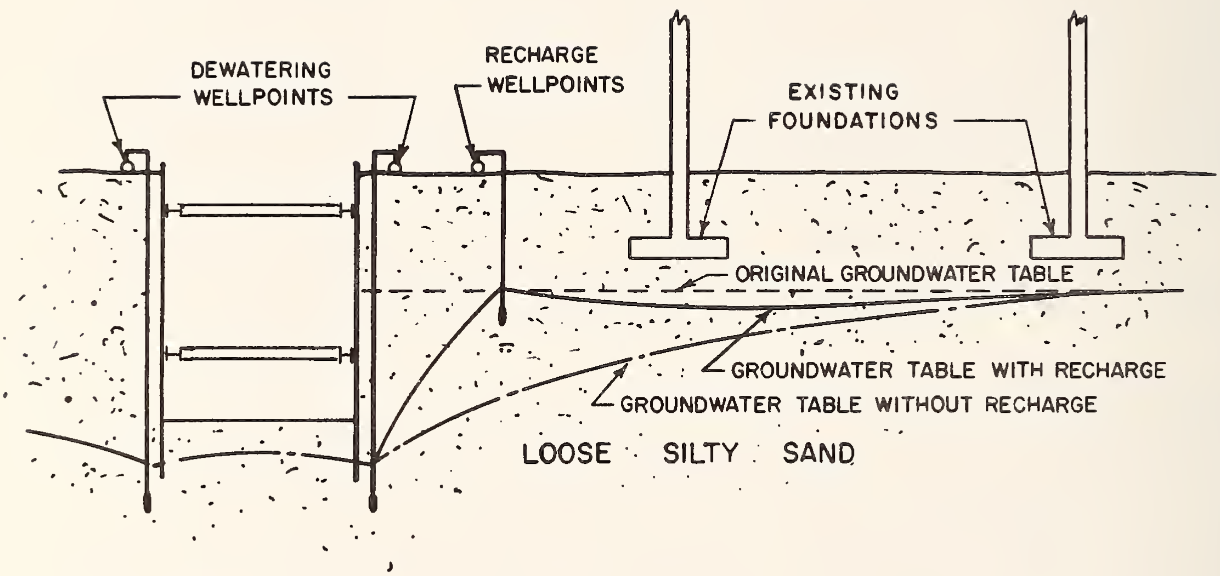


FIGURE 14 - Recharge of Groundwater to Prevent Settlement (Ref. 173)

Recharge can be accomplished by several means, including surface recharge basins, ditches, wells, and wellpoints. Horizontal perforated pipes are occasionally used. The procedure of returning water to the ground is more difficult than removing it, primarily because of clogging or encrustation problems. Ions which are normally soluble in groundwater in the natural state may precipitate due to pressure drops or aeration which result from pumping. This precipitate is continually purged from a dewatering system, and when encrustation does occur, it may be cleaned effectively with acid and the encrusting deposit removed by pumping after cleaning. If this water is to be used for recharge, however, these insoluble precipitates will accumulate at the injection location and decrease discharge capacity due to clogging of the filter material or screens. The bottom of recharge basins must be scraped occasionally for the same reason to ensure good infiltration rates. If well or wellpoint systems are used for recharge, they must also be cleaned periodically.

Recharge systems are not continually purged. Any removal of encrusting agents must be accomplished by temporarily pumping the system for backflushing. This technique refers to subsurface recharge techniques only. When the build-up of these insoluble precipitates is very rapid, it may be necessary to treat the water chemically prior to injection. Water conditioners can be used to keep the problem ions in solution to avoid this build-up. Chlorine may be added to disinfect wells and prevent the formation of ion deposits of bacterial origin. Rapid sand filtration may also be employed in some instances for pretreatment of recharge water.

Another problem inherent in operating recharge wells or wellpoints is that of entrained air. Air accumulation can cause distribution pipes to choke, or a build-up of air trapped inside the injection well can depress the water level below the driving head necessary for adequate recharge. Another problem is that air bubbles may enter the pores of the aquifer, thus lowering its permeability, and resulting in decreased flows. Therefore, the air must be removed at separating chambers prior to reaching the recharge point. These chambers typically consist of a tank with an air/water interface under slight vacuum to encourage degassing. All other piping must be airtight after leaving the separation chambers. Even with these precautions, temperature changes can cause release of dissolved gases in the water, a design consideration not to be overlooked.

If the rate of pumping into a recharge system is constant, reduction in capacity of a recharge well leads to a rise in water level in and around the well to such an extent that surface seals can be breached. Uncontrolled overflowing water at the surface must be controlled to prevent flood damage, particularly in urban areas.

The effects of recharge systems on nearby dewatering systems must be considered. If the recharge system is near the pumping system, it usually necessitates additional pumping capacity. In extreme cases, it may require a completely different groundwater control concept. The combined costs of the recharge-dewatering systems must be evaluated against other methods which do not require recharge. If the recharge system is placed more than twice the radius of influence from the dewatering system, it should have no effect on the construction; however, the increased costs of transporting the water must be evaluated.

Occasionally, municipal water is used for recharge; however, because scale deposits inside city pipes may break loose due to increased flows, serious clogging can occur. This is most evident after major flow disturbances such as fire flows or repairs of leaks in mains. Also, the release of air and dissolved gases must still be anticipated.

A system of 15 recharge wells was used in New York City to recharge a portion of the flow from a dewatering system for construction of a section of subway. The purpose of the system was to recharge the water into an aquifer also being pumped for water supply. The groundwater rights were owned by a water company which charged the owner for the net water removed. It was determined that the cost for installing a recharge system would be less than the cost of paying for recharge water. The recharge system was approximately 3,000 feet (915m) from, and aligned perpendicular to, the dewatering system. Since the radius of influence of the pumping system was estimated to be 3,500 feet (1,070m), an additional dewatering well was required for predrainage, together with high-head pumps in all the wells to push the water to the recharge area. Unexpected problems with iron in the water required cleaning of the system at more frequent intervals than planned. To minimize these costs, wells were allowed to deteriorate to such an extent that several of the wells overflowed, causing nuisance flooding of the area.

A recharge system was also installed during construction of an intercepting sewer tunnel near the Brooklyn Bridge in New York City. The tunnel was driven under compressed air; however, a system of wells was installed to dewater the shaft and turnunder which was capable of lowering the water level 10 feet (3m) under an anchorage for the bridge. Because the anchorage was constructed on a timber grillage which might have

deteriorated if not submerged, recharge wells were installed around the anchorage to maintain water levels. The water was supplied from a city water main, and periodic cleaning was required to remove iron deposits which accumulated slowly.

4.40 CUTOFF WALLS AND TRENCHES

Intercepting horizontal water flows by means of cutoffs is a common technique in open excavations, cofferdams, and earth dams. These techniques are not commonly applied to bored tunnels, but are frequently used on cut-and-cover projects. Procedures may involve steel sheet piling or slurry trenches, which may be temporary barriers and support systems, or elements of the final structure, i.e., slurry walls. For a cutoff wall to be effective, it should either intercept an impervious layer or extend deep enough to reduce flow quantities and exit gradients. The judicious use of cutoffs should be considered with a full understanding of the applicability and limitations of each type as discussed below.

It is important to recognize that cutoffs are a passive means of groundwater control. For example, surrounding an excavation with a cutoff may not eliminate the need for dewatering (pumping) within the cutoff to drain the stored water and stabilize the volume of soil enclosed by the cutoff. Barrier techniques, such as cutoffs, grouting, or freezing, will usually greatly reduce active dewatering requirements but not necessarily eliminate them completely.

4.41 Steel Sheeting

For temporary dewatering, steel sheet piling is effective and economical in a variety of situations. A number of shapes and sizes are available within the U.S. and abroad to accommodate anticipated soil conditions. (Ref. 109) The primary advantages of sheet pile cutoffs are their ease of installation, ability to withstand high driving stresses, long-term durability, and reuse potential.

The effectiveness of sheet pile walls with respect to reduction of flow is greatest in relatively pervious soils, as opposed to silty or clayey sands, where the permeability of the sheet pile system can be near that of the surrounding soil (Ref. 108). Effectiveness is also dependent upon proper penetration into an impervious stratum and the condition of the sheeting elements after driving. Insertion into material which is very dense or which contains boulders or obstructions may result in ripping of the sheeting or damage to the interlocks and subsequent reduced effectiveness, as illustrated in Figure 15.

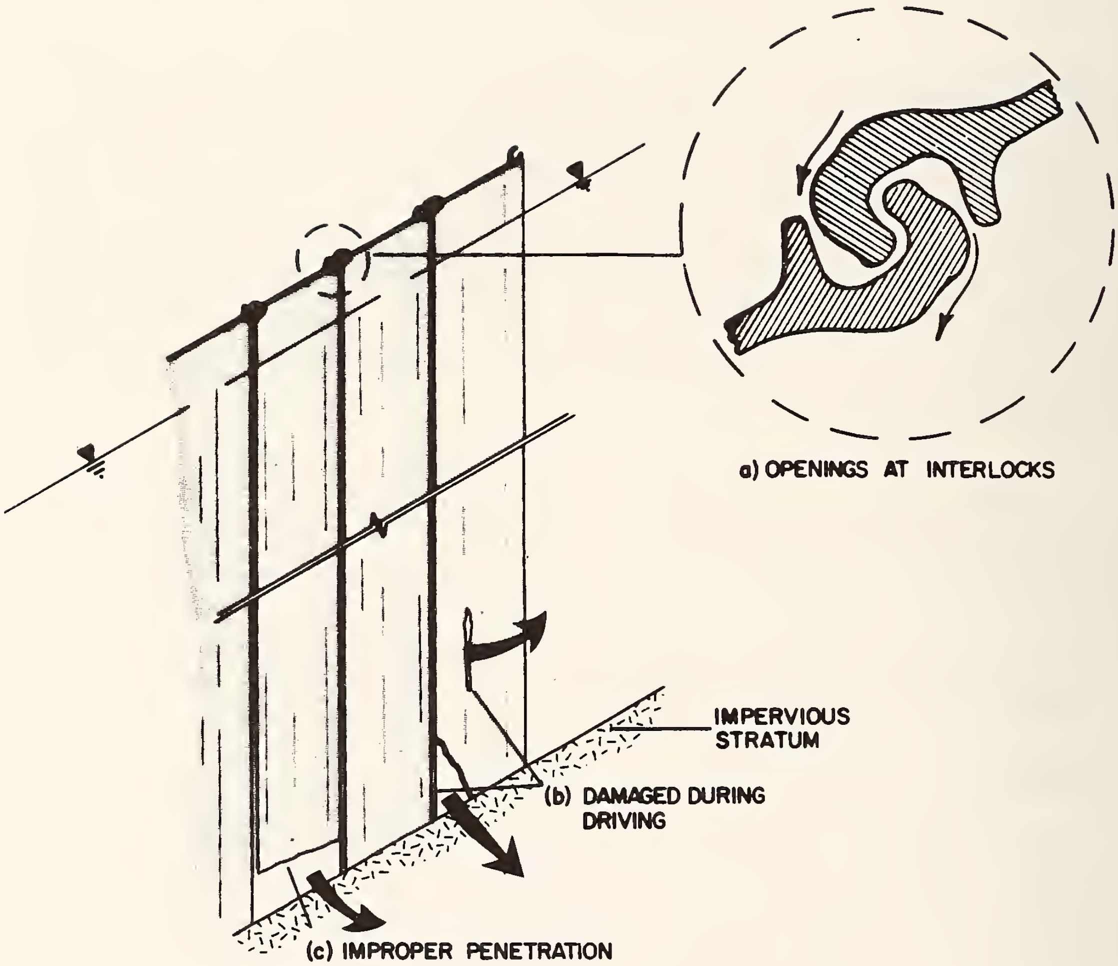


FIGURE 15 - Sources of Leakage Associated with Steel Sheet Cut-Offs

Cutoff efficiency has been studied in relation to the degree of imperfection of the wall and the degree of penetration into an impervious stratum (Ref. 262). Studies of typical sheet piling sections indicate that spaces of 1-2mm wide can occur at the interlocks in loosely fitting sections and correspond to nearly 0.5% of the total area of sheeting. Even at this low level of imperfection, the effectiveness of the cutoff is substantially reduced. Additional openings in the sheeting caused by driving in difficult ground conditions can further decrease cutoff efficiency and reduce it to zero when the degree of imperfection surpasses 1%. Under normal site conditions, sheet pile effectiveness can be increased by reducing the gap width at the interlock, a process which occurs as the wall is stressed by lateral earth pressure. Efficiency can also be increased if interlock openings become filled with soil during installation or through the migration of fines after steady-state flow has been established. Examples of increased cutoff efficiency beneath dams has been attributed to the eventual piping of fine materials into the narrow openings in sheet pile walls.

The degree of sheeting penetration will affect cutoff efficiency to a large degree. Whereas full penetration to an impervious stratum will be effective in cases where the degree of imperfection is low, partially penetrating sheeting may lead to large gradients beneath the toe and subsequent piping or heave in adjacent cuts. Consideration should also be given to possible damage to the sheeting from driving into very hard impervious material such as glacial till.

4.42 Slurry Trenches and Diaphragm Walls

The slurry trench technique refers to the use of a thick bentonite slurry to support the sides of excavated trenches. The bentonite fluid can be mixed with cement or excavated soil to form a temporary watertight structure, or the slurry can be displaced by precast or tremie concrete to form permanent structural walls. With respect to tunnel dewatering, both temporary and permanant cutoffs using slurry methods are applicable, usually in cut-and-cover applications. A number of variations on the slurry technique have been developed to accommodate a variety of possible field requirements for wall depth, wall stability, watertightness, etc.

The use of bentonite in slurry trench construction is a result of the thixotropic properties of a properly mixed bentonite suspension. The tremendous swelling abilities of sodium bentonite give rise to a slurry which is highly fluid when mixed, but which exhibits a significant increase in viscosity when allowed to set. The specific gravity of the slurry in concentrations of 4-5% results in horizontal pressures which are capable of maintaining open trenches and holes to large depths (Ref. 138).

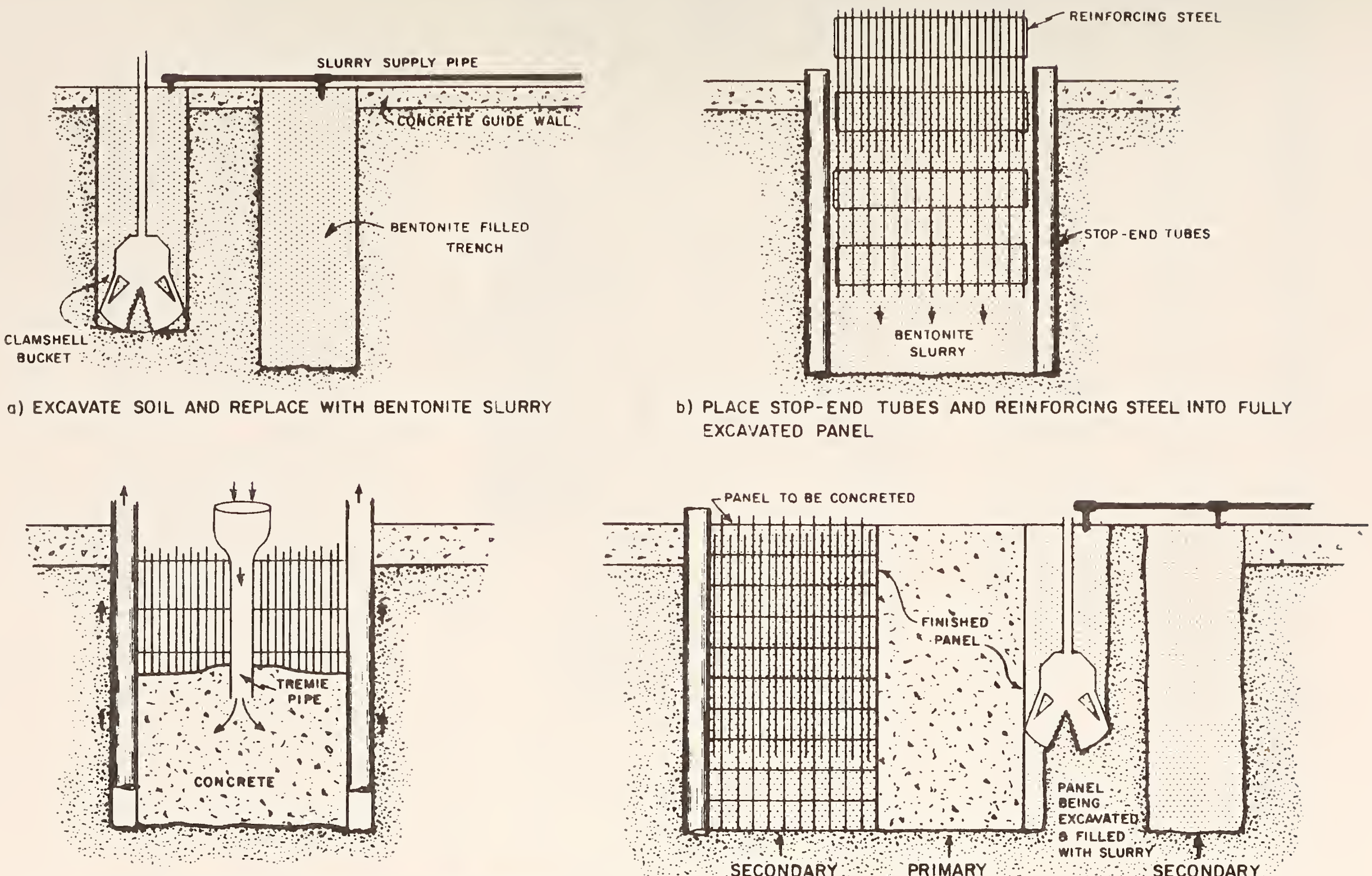
In addition, these horizontal pressures result in the formation of a filter cake along the sides of the hole which helps to maintain the trench stability and inhibits the infiltration of slurry into the adjoining soil. This filter cake is thought to be the main factor controlling the permeability of slurry trench cutoffs.

4.43 Cast-in-Place Slurry Wall

Use of the conventional slurry wall is generally restricted to cut-and-cover tunnel excavations. The wall may or may not be designed as part of the permanent structure, although recent trends are to do so. Probably the most extensive use of slurry walls as part of the permanent tunnel structure is for the MBTA Red Line extension through Cambridge and Somerville, Massachusetts. Two major cut-and-cover stations--Harvard Square and Davis Square and 1.3 miles (2.1 km) of cut-and-cover line section--Davis Square to Alewife Station--will employ slurry walls as part of the final structure.

The procedure is described (Figure 16) in many places and basically consists of a narrow trench excavated using a modified clamshell type bucket or backhoe and kept open with a slurry (Refs. 57, 108, 283). Reinforced concrete guide walls constructed at the surface aid the excavation equipment and stabilize the top of the trench against caving. Hollow stop-end tubes, which are the most common joint forms, are placed at the ends of each excavated panel to provide a uniform joint between adjacent sections. A prefabricated reinforcing steel cage is placed in the excavation and concrete is tremied into the cut displacing the slurry. Alternate panels are constructed using this procedure and the stop-end tubes are removed before the concrete has set completely. Secondary panels are then excavated between the primary walls using a similar procedure, with the section completed after the setting of steel and pouring of the tremie concrete. Typical panel dimensions measure 1-3 feet (0.6-0.9m) wide by 10-20 feet (3-6.1m) long. Dimensions are usually limited by the size of the crane available.

During the construction of conventional panels, the slurry is usually maintained at a level of 3-5 feet (0.9-1.5m) above the groundwater level to ensure trench stability. In situations where the groundwater is high or artesian pressures are present, special measures, such as dewatering, may become necessary (Ref. 57). Other difficulties which may be encountered typically involve the existence of obstructions within the trench such as boulders or fill material. To remove these obstacles often requires the use of percussion or rotary drills which reduces progress and may result in irregular trench walls (Ref. 84). There have been a number of variations on the basic cast-in-place slurry trench method. Among these are:



a) EXCAVATE SOIL AND REPLACE WITH BENTONITE SLURRY

b) PLACE STOP-END TUBES AND REINFORCING STEEL INTO FULLY EXCAVATED PANEL

c) POUR TREMIE CONCRETE TO DISPLACE SLURRY, REMOVE STOP-END TUBES

d) DIFFERENT CONSTRUCTION PHASES

FIGURE 16 - Schematic of Conventional Cast-in-Place Slurry Wall

1. Continuous construction involving the excavation and concreting of successive panels using a single stop-end tube.

2. The soldier pile and tremie concrete method (SPTC) involving the use of driven or placed soldier piles with panels excavated in between using bentonite slurry.

3. The use of bored secant or tangent piles consisting of augering under a slurry and subsequent concreting of alternate holes; the intermediate holes are then augered and concreted to form a continuous cutoff. Typical hole diameters range from 12-16 inches (30-40cm) to 2.5-4 feet (0.8-1.2m).

For waterproofing applications, the major drawbacks of cast-in-place structures involve leakage through construction joints at the interface between panels. This leakage occurs mainly where stop-end tubes were used in the construction sequence. Common methods of improving the integrity of construction joints include keyed joints, waterstop joints, or grouting (Ref. 84, 57). Seepage through the concrete itself is usually not a major concern. (See Volume 2 for a thorough discussion of leakage considerations.) Factors which affect the seepage through concrete walls include the concrete curing environment, the degree of infiltration of the bentonite slurry into the adjacent soil, the construction procedures and concrete integrity, and wall movement after excavation (Ref. 248). A complete discussion of proper concrete placement techniques is presented in Volume 2. Since concrete slurry trench walls are used almost exclusively for cut-and-cover tunnels, areas of obvious seepage can be easily detected after excavation and remedial measures taken.

4.44 Precast Slurry Panels

As with the cast-in-place concrete walls, precast walls in slurry trenches are usually used in cut-and-cover operations where the wall is to be used as part of the final structure (Figure 17). The construction procedure consists of excavating the trench using a special cement-bentonite slurry, inserting precast panels into the slurry, and suspending them in the trench until the slurry hardens. Adjacent panels are typically connected using tongue-in-groove connections, T-Beams and slabs (Panosol system) or hooks and bars (Prefasif System). When the cement-bentonite slurry hardens enough to support the precast panels, the material on the inside of the walls can be excavated and the wall braced as in conventional slurry wall construction.

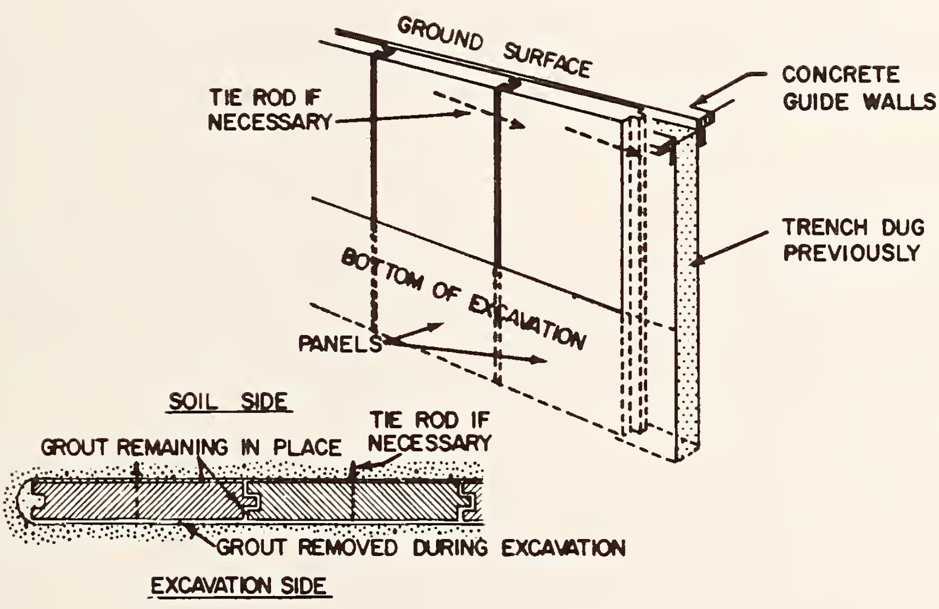


FIGURE 17 - Precast Slurry Panels
(Ref. 84)

The hardened slurry on the inside wall face is stripped away, while the slurry on the soil side of the panels and at the joints acts as an impermeable barrier to prevent seepage. Additional watertightness is obtained through the use of waterstops or grouts in the voids between panels.

It is reported that precast panels can be installed as economically as cast-in-place walls if they can be made on-site (Ref. 84). Structural and aesthetic advantages include high-quality concrete, well-placed reinforcing, accurate panel alignment within the trench, and a high-quality finish face on the wall. Trench excavation and panel placement is continuous and thus requires careful control of the slurry properties. Wall depths are limited by the height of the precast panels, but are usually 30-50 feet (9.1-15.2m).

The watertightness of the precast slurry walls is usually better than that of the cast-in-place walls because of the careful sealing of panel joints and outside wall face with hardened slurry. For the construction of cutoffs in thick strata of pervious soils, composite walls can be constructed consisting of deep trenches backfilled with impermeable slurry beneath precast structural panels (Ref. 84). In all cases, the use of precast panels for dewatering should be limited to those cases where the wall is to be used as a part of the completed structure, i.e., cut-and-cover tunnel construction. Dewatering alone, such as for bored tunnels, is achieved much more economically using other cutoff or predrainage methods.

4.45 Slurry Trenches

For nonstructural applications, trenches backfilled with mixtures of bentonite cement or bentonite and soil slurry can provide effective cutoffs. In extreme conditions, a bitumen backfill may be used to increase cutoff deformability (Ref. 57). As opposed to the rigid properties of concrete diaphragm walls, the use of self-setting "plastic" backfill results in a low-strength, but impervious, cutoff wall which has very little load-bearing ability. These methods have not been widely used in connection with tunnel construction but are worthy of consideration in special situations.

Construction of the self-setting slurry trenches is usually a single-stage process consisting of trench excavation with a clamshell or backhoe and concurrent backfill with a slurry. The excavation sequence depends upon the construction method and type of backfill used. Where a cement-bentonite slurry is used as the backfill material, excavation may proceed in a continuous trench.

Where a soil-bentonite backfill is used, the excavation is usually performed in alternate panels, as depicted in Figure 18. A single panel is excavated under a bentonite slurry. As the trench is dug, the excavated soil is stockpiled. When the full panel depth and length have been achieved, the excavated soil is mixed with more bentonite slurry and placed in the trench, displacing the stabilizing fluid. The resulting cutoff is highly stable due to the suspended solids and economical due to the low volume of bentonite needed.

The dimensions of backfilled slurry trenches depend upon the type of backfill, thickness of the pervious stratum, groundwater conditions, and excavation techniques. Reported depths of up to 250 feet (76m) are possible with clamshell and kelly bar rigs, 100 feet (30m) with draglines, and 35-40 feet (10.7-12.2m) with backhoes. Slurry trench widths vary with the ability to control the backfill placement; where cement-bentonite backfill is well mixed, good placement is assured, and the wall is thin (2-3 feet, 0.6-0.9m); whereas possible heterogeneous placement of a soil-bentonite backfill usually mandates wider trenches (up to 10 feet, 3m) to ensure proper placement.

4.46 Thin Wall Cutoffs

Although the construction of thin wall cutoffs does not utilize the slurry technique per se, it is often described as a slurry method because the resulting cutoff is composed mainly of a bentonite fluid. The basic procedure involves the insertion of steel H-pile into the ground by driving or vibration, with concurrent injection of slurry through tubes welded to the web. When the pile reaches the desired depth, it is extracted, while a grout is pumped to fill the remaining voids. There are two commonly used procedures for forming the cutoff wall. The first involves continuous insertion of a single pile, as shown in Figure 19a; each insertion of the structural member typically overlaps the previous one 4-6 inches (10.2-15.2cm). The second involves the use of a series of piles where the members withdrawn at the end of the section are then redriven at the head of the line; this procedure is depicted in Figure 19b. The method is applicable in any pervious soil into which injection beams can be driven or otherwise inserted.

An advantage of the thin wall cutoff stems from savings in excavation and slurry costs. Cutoff walls constructed using this technique are often very thin (minimum thickness of 1-4 inches, 2.5-10.2cm) and irregular in shape, but are capable of providing a good quality impervious barrier (Ref. 233). Although the procedure has not been used extensively in the United States, its success in Europe is well known. The type of injection beam and slurry mix used can be varied with the site conditions and permeability requirements.

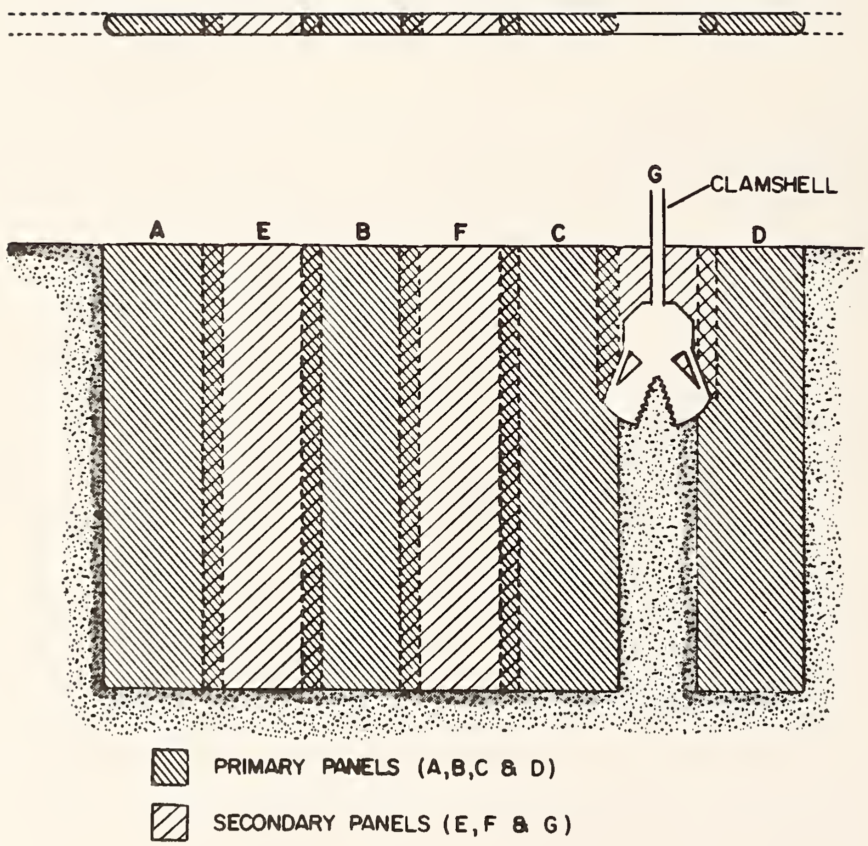


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FIGURE 18 - Slurry Trench Panel Sequence (Ref. 283)

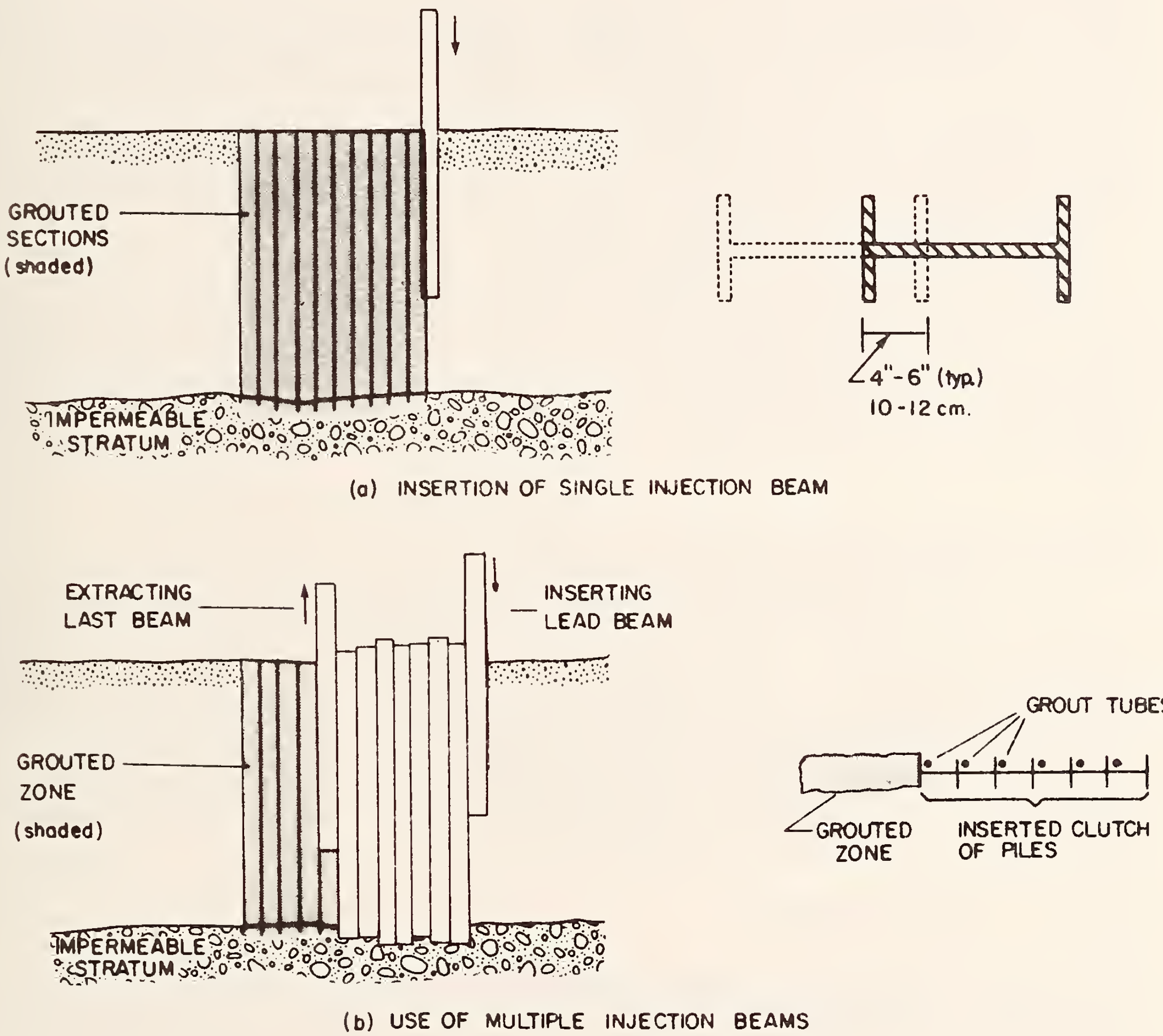

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FIGURE 19 - Injection Beam Grouting (Ref. 57)

4.47 Considerations and Limitations of Slurry Wall Techniques for Tunnel Dewatering

Requirements for dewatering during cut-and-cover tunnel construction can be satisfied in conjunction with permanent dewatering requirements through the use of rigid concrete diaphragm walls constructed using a number of previously described techniques. In general, these walls act not only as seepage barriers, but also as lateral soil support systems during construction and as permanent tunnel walls after construction. Groundwater control is usually very effective, although seepage problems may arise under certain conditions, some of which are discussed below.

4.47.1 Improper Wall Penetration

Large reductions in cutoff efficiency and high gradients can occur if the pervious stratum is not completely penetrated. Theoretical and experimental evidence has been reported showing that penetration to 95% of the depth of the pervious layer will reduce cutoff efficiency by 50% for certain geometries (Ref. 262).

4.47.2 Poor Construction Joints

For cast-in-place concrete walls or bored secant pile walls, poor contact between adjacent panels or piles may result in water seepage. This problem can occur if slurry flows around stop-end tubes or if corners of concreted panels become broken. As these defects become apparent, they are usually easily repaired by cleaning and sealing or grouting. As would be expected, frequent joint spacings invite more difficulties than do wide spacings. Precautionary waterproofing methods include keyed joints or water stops installed between panels.

4.47.3 Poor Concrete

Although the tremie method of concreting is usually reliable, contamination of the mix may result from improper placement or accidental dilution with water. In addition, segregation or poor mixing may reduce the integrity of the concrete. The resulting weak zones within the wall are more susceptible to degradation and erosion by water (Ref. 283). Another source of poor concrete is an excess of reinforcing steel, where proper displacement of the bentonite slurry is not guaranteed.

4.47.4 Differential Wall Movement

Relatively small movements between adjacent panels of diaphragm walls can cause openings to develop. The movements can be caused by earth pressure, inadequate tiebacks or, differential

settlement of panels. It is reported that wall movements causing openings are more likely to occur at wall corners than in other areas (Ref. 249). Shear connectors can help to reduce differential movement, although it has been recommended that a more practical approach involves repairing leaky joints as they appear (Ref. 248).

4.48 Considerations and Limitations of Slurry Trench and Thin Wall Cutoffs for Tunnel Dewatering

Slurry trench cutoffs and thin wall cutoffs are suited for applications near bored tunnels or as secondary cutoffs near cut-and-cover excavations. These methods provide the most economical means of dewatering using slurry methods because of the ease of construction and inexpensive materials. They have been proven effective as dam cutoffs, reservoir seepage barriers, sewage and chemical waste isolators, and in the dewatering of deep excavations (Refs. 34, 57, 133, 168, 220). The plastic nature of the backfill allows the cutoff to undergo large movements without rupturing; nevertheless, problems can develop under certain circumstances:

4.48.1 Improper Wall Penetration

Insufficient depth of the trench to an impervious stratum will result in the problems previously outlined.

4.48.2 Insufficient Mixing of Backfill

In soil bentonite backfills, poor mixing can result in pockets of pervious material (sand, gravel, etc.) which will form "windows" within the finished wall. This can also occur if the slurry is too thick to allow proper mixing of its constituents.

4.48.3 Slurry Contamination

Salt or calcium in the groundwater may render the bentonite slurry useless or give undesirable characteristics in terms of viscosity, set-time, or filter cake formation.

4.50 GROUTING

Grouting has been used extensively in Europe and Great Britain for the stabilization of soils and rock and is a common construction technique. In the United States, grouting in tunnel construction has generally been reserved for emergency situations and cases where remedial treatment is necessary to reduce large flows of groundwater or stabilize caving soils. Only recently has this process been used as a primary construction technique for reducing water flows, minimizing soil movements, and reducing

air loss or blowouts in tunnels excavated by pneumatic shield methods. It is anticipated that grouting techniques will find wider application with U.S. tunnel designers as their familiarity with the techniques increases.

Effective control of groundwater around tunnels and open cuts can be achieved through grouting of soil and rock. Grouting is the injection of fluids into soil voids or rock fissures and cavities for the purpose of altering their physical properties. Most often these fluids are cement, clay, sand-cement, clay-cement, or chemical mixtures which are pumped into the medium in a fluid state and later harden or gel, significantly altering the strength and permeability of the soil or rock mass.

The effectiveness of grouting is a function of the subsurface conditions, grouting method and material, and "groutability" of the strata. A wide selection of grouts allows for the treatment of all pervious soil and rock profiles. There are many types of grout and injection techniques. The major ones will be discussed in the following sections.

4.51 Grouting Methods and Procedures

The extent of grouting programs is highly variable depending primarily upon the subsurface conditions. A thorough subsurface investigation is therefore necessary.

Three common approaches to grouting for groundwater control during construction of tunnels are surface grouting, grouting ahead of the bore, and grouting from shafts or pilot tunnels. When tunnel machines and liners are used, grout is usually pumped into the voids between the tunnel wall and the liner system to reduce ground movements and water migration behind the liner.

The easiest approach is surface grouting where vertical or inclined grout holes are drilled from the surface to stabilize a soil stratum at depth prior to tunnel excavation. A typical application in Vienna is depicted in Figure 20. This procedure is generally fast and does not suffer the inconveniences of space limitations which hamper grouting from within the tunnel. Although this is the most commonly used method for tunnel grouting, its use is limited in very deep tunnels (depth > 40 feet, > 12m) or when tunneling through urban areas. In these cases, grouting is often accomplished from within the excavated tunnel.

The process of "grouting ahead of bore" is typically used on deep tunneling projects or when tunnels traverse under bodies of water where surface grouting is impractical. The procedure involves grouting in a pattern of concentric circles around the face of the tunnel to a horizontal distance of 15-25 feet (4.6-7.6m), and then excavating through the grouted material

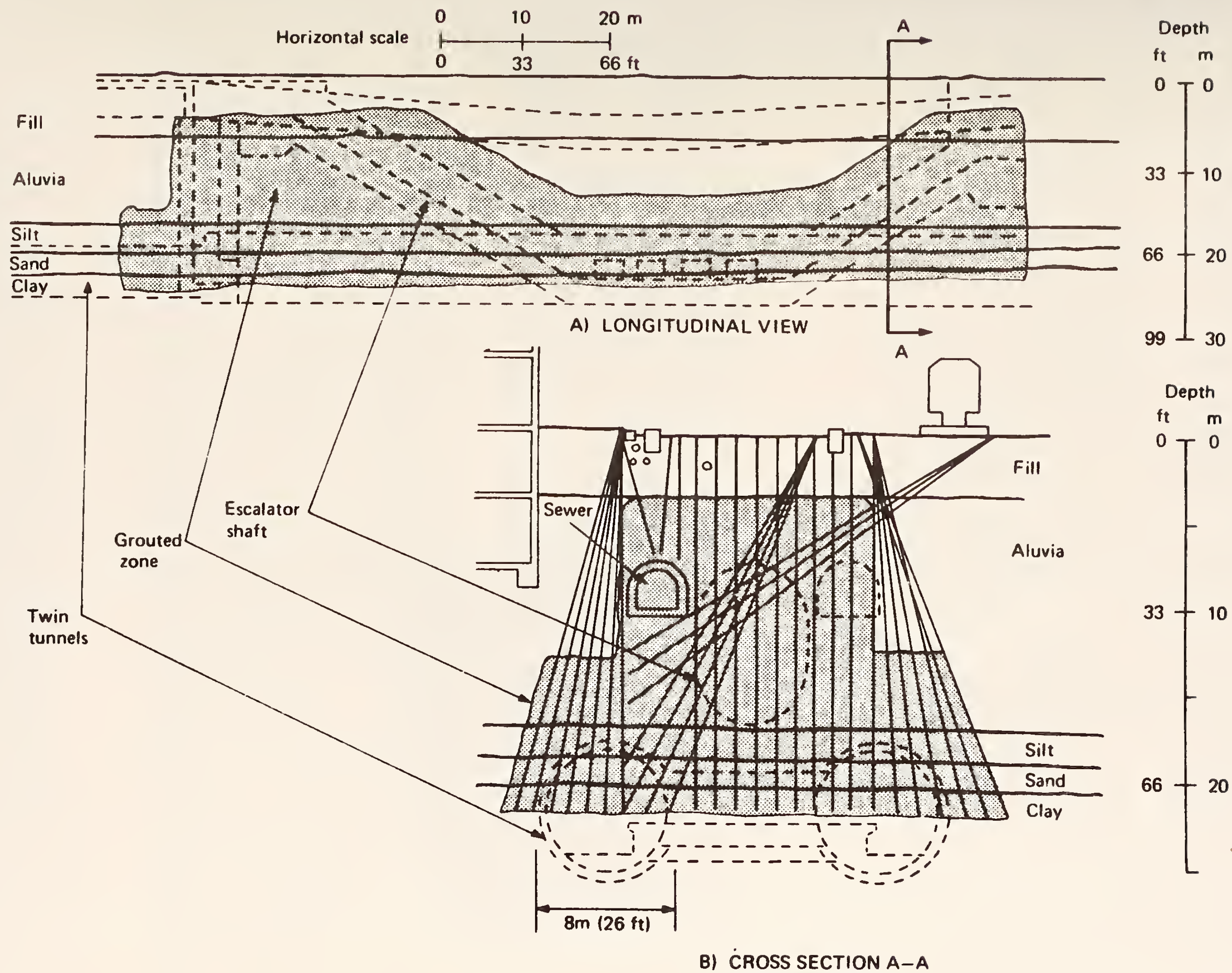


FIGURE 20 - Grouting Scheme, Karplatz Station, Vienna (Ref. 201)

until the ungrouted zone is neared. Figures 21 and 22 illustrate possible schemes. As excavation proceeds, additional grouting can be performed at intervals along the crown, sides, and invert of the tunnel to stabilize the soil or rock and fill voids between the lining and the tunnel wall. This procedure is slower than surface treatment due to space limitations.

A third grouting procedure which is normally used on large projects is the use of shafts or pilot tunnels (Auber Street Station, Paris, France; Blackwell Tunnel, England). Grouting is performed from cased shafts sunk to the desired depth or from pilot tunnels excavated in dry, competent material. Grouting is performed in a radial pattern from the shaft or tunnel, as illustrated in Figure 23 for the Auber Street Tunnel. These excavations have the disadvantage of limited space for drilling and grouting, but the treatment can take place concurrently with the main tunnel excavation with minimal disturbance of surface operations.

4.52 Drilling/Grouting Techniques

Grouting can be achieved through a number of conventional techniques, depending on the location of the project and experience of the contractor and engineer. In general, 1-1/2- to 3-inch diameter (3.8-7.6cm) grout holes are driven to the desired depth, washed under pressure to remove cuttings and loose material, and then tested under air or water pressure to determine permeability or infiltration rates. The data from these pressure tests are used as an aid in the choice of grout type and grout consistency, although the basic type, i.e., cement or silica, etc., is normally determined well in advance.

During the grouting operation, continuous monitoring of the process is necessary. The monitoring includes constant observation of grout pressures, grout consistency, and volume and rate of grout intake. This information is often used to determine necessary modifications in the grouting procedure, such as changes in grouting pressure or consistency or termination of the grouting due to soil fracture or heave.

The most effective grouting procedures may often be chosen by the contractor or engineer from local experience or observations as the grouting proceeds. The most commonly employed techniques are:

4.52.1 Stage Grouting

Stage grouting is grouting of a borehole by segments in either ascending or descending order. The former method involves drilling to the full depth of the hole and then washing, testing, and grouting individual segments through the use of

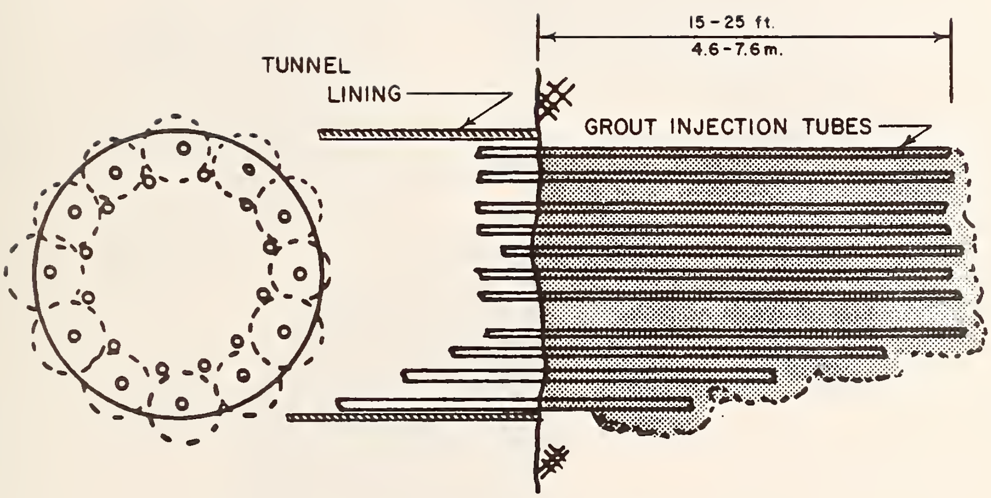
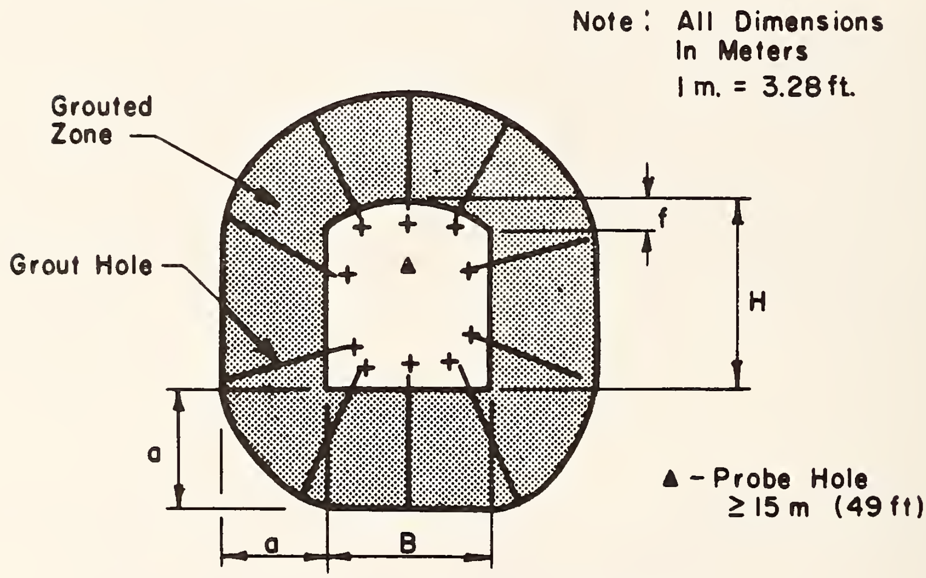
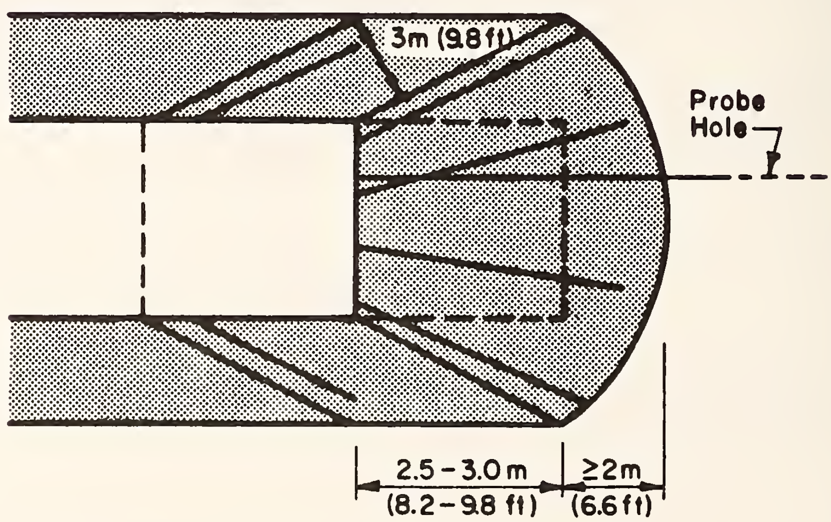


FIGURE 21 - Schematic Injection Pattern
for Tunnel Face Grouting (Ref. 133)

Type	H	B	f	a
Station Tunnel	6.75	9.20	≥1.40	3.50
Double Track Tunnel	6.00	8.10	≥1.60	3.50
Single Track Tunnel	5.60	4.30	≥1.00	3.00



a) Transverse Cross-Section



b) Longitudinal View

Figure 22 - Rock Grouting Pattern for Tunnels Stockholm Underground (Ref. 20

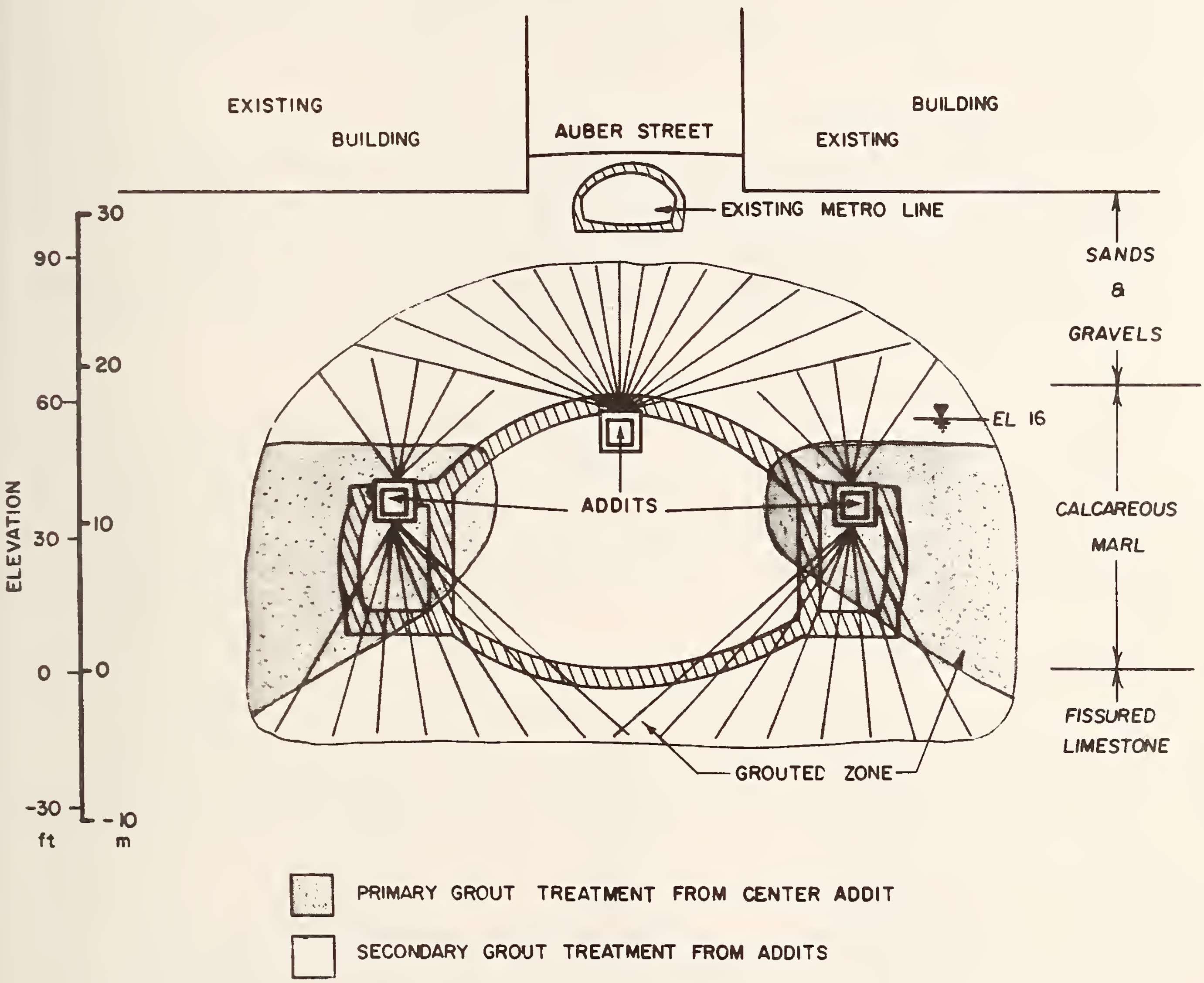


FIGURE 23 - Grouting from Galleries
Auber Station, Paris Metro (Ref. 133)

packers. Stage grouting in descending order requires drilling to a certain depth, washing, testing, and grouting, and then redrilling to the next depth and repeating the process without the use of packers. Although moderately expensive, this method allows for individual treatment of each segment of the hole and increasingly higher pressure.

4.52.2 Series Grouting

Series grouting involves drilling a series of grout holes to a certain depth and grouting them all. The grout holes are extended by boring through the set grout to a new depth and repeating the process. This approach allows for higher grout pressures to be used for deeper foundations without problems of grout rising to shallower depths and causing cracks or heaving.

4.52.3 Circuit Grouting

Circuit grouting is a procedure by which grout is conveyed to the bottom of a borehole through a pipe with the material not taken by the stratum returning to the surface in the annular space between the injection pipe and the borehole wall. The grout which returns to the surface is filtered and recirculated down the injection pipe. This procedure minimizes segregation of coarse-grained grouts through constant movement and agitation.

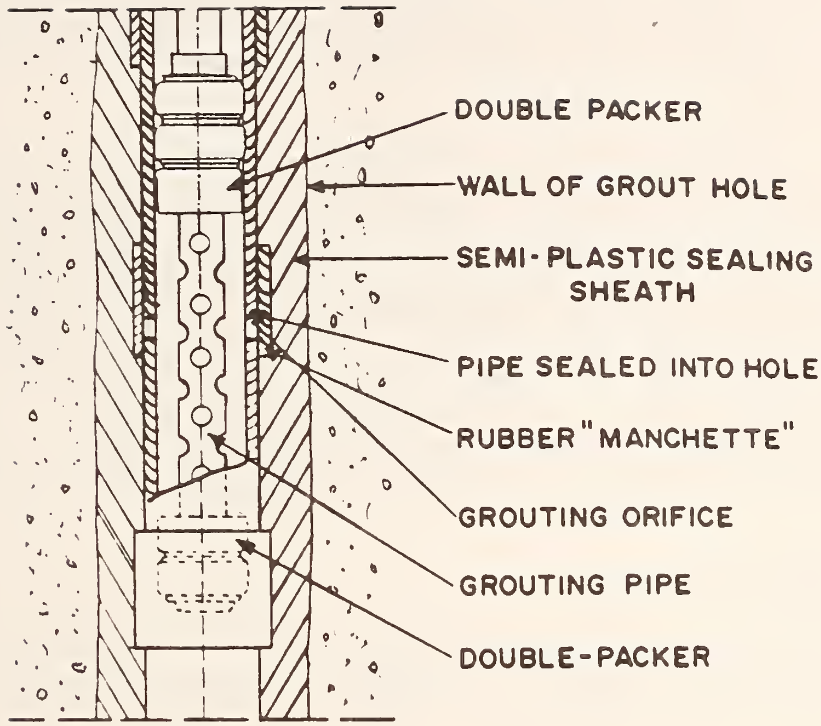
4.52.4 Packer Grouting

Packer grouting is the cheapest and fastest technique available. The packer or "stop" grouting method can be used to grout small sections of a borehole from bottom to top, allows for accurate control and monitoring of the grout intake at each level of the borehole, and allows for grouting at elevated pressures.

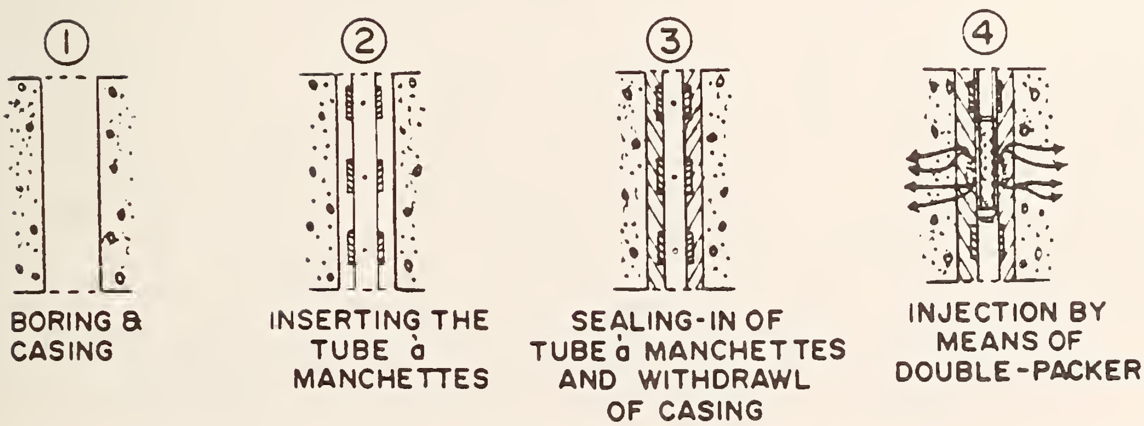
4.52.5 Tube-a-Manchette

Tube-a-manchette is a procedure used extensively in Europe and Great Britain and is a versatile and quick method of grout injection. Figure 24 depicts the method. This procedure allows for good control of grout pressure and grout take at various depths, and permits regrouting of specific zones. It is more expensive than conventional grouting procedures.

Independent of the method chosen to grout a certain soil profile, grouting must be performed until the area is properly stabilized against the flow of water. For surface grouting, this is usually accomplished by "split spacing" a group of grout holes. That is, grouting a set of "primary" holes spaced at intervals of 20 to 40 feet (6.1 to 12.2 meters) then grouting



(a)



(b)

FIGURE 24 - Operational Principal of Tube-a-Manchette (Ref. 133)

a set of secondary holes at intermediate locations, and so on until the zone will not take any more grout (Figure 25). Grouting from within the tunnel proceeds in a similar manner with successive circular patterns of grout holes driven into the tunnel face (Figure 21).

Determination of adequate grouting is largely based on the experience of the contractor. Pumping of less viscous grouts (chemical grouts) after the primary treatment, and pressure testing of a hole after the primary grout has set, are common ways of testing for adequate grouting. A rule of thumb recommended by the U.S. Corps of Engineers states that the hole is adequately grouted when the rate of grout take at the maximum allowable grouting pressure is less than one cubic foot within a period of 10 minutes (U.S. Corps of Engineers Manual EM 1110-2-3501). Common European practice incorporates the use of measurements of surface heave to indicate a successful grout program (Ref. 201).

4.53 Types of Grout

Available grout materials can be classified as suspension grouts, chemical grouts (including resins, gels, and colloidal solutions), and emulsions.

Suspension grouts are the most common and include coarse grouts which contain particles in suspension such as cement, sand-cement, clay, clay-cement, etc. These materials are usually the most viscous of the available materials, which limits their applications to the grouting of rock or coarse soils. They are typically the cheapest of the presently used materials.

Chemical grouts rely on chemical reactions to form hardened gels. They initially have low viscosities and are more versatile than suspension grouts, yet considerably more expensive. These grouts are commonly: 1) silicate based materials which react with an organic reagent; 2) chrome-lignin grouts; 3) acrylamides; or 4) resins. Chemical grouts are widely used in Europe on a regular basis as solutions to potentially hazardous groundwater conditions (Ref. 71). The low viscosity of most chemical solutions and the accuracy with which set time, strength, and permeability can be controlled have made these types of grouts tremendously useful in finer grained cohesionless soils and as a secondary treatment for grouting of coarse soils and rock fissures.

Bituminous emulsions are used where other grouts may be washed away by groundwater action before they have time to set. Careful control of emulsion ingredients is necessary to ensure proper coagulation of the bitumen once it enters the soil or rock openings.

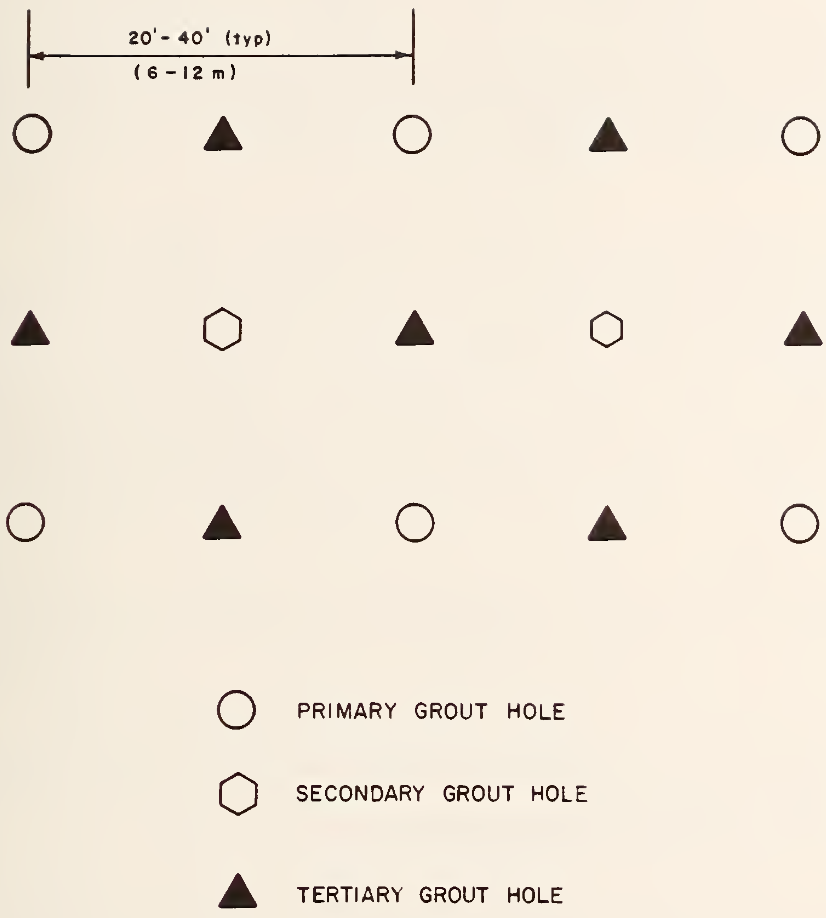


FIGURE 25 - Schematic of Split Spacing Surface Application (Ref. 76, 88, 89, 91, 104, 133, 140)

The following sections contain concise summaries of the major types of presently available grouts and their applications to the dewatering of tunnels.

4.53.1 Cement Grouts

The use of cement mixed with any one of a number of additives forms an inexpensive suspension grout capable of infiltrating medium- to coarse-grained soils or small rock fissures (Ref. 44, 104). Portland Type I cement is the most common type used, although Types II and III have been used in special situations (high sulfate resistance or high-early strength). Air entrainment procedures are not recommended for cement grouts.

Cement is seldom used alone in grouting, but is mixed with clay or sand, rock flour, loess, straw, fly ash, silt, or diatomite for increased economy. The addition of each material will alter the properties of the grout mix in a different manner. Most often the additives will reduce the grout strength but may increase its waterproofing capabilities. For example, the addition of either natural clay or bentonite will reduce segregation tendencies of the grout and is ideal for primary treatment of coarse-grained soils or rock fissures. Typical reported cement:clay:water mixes are 2:(2-3):(3-6) (Ref. 165), although other proportions are reported (Ref. 43).

The addition of sand to cement mixes has been widely used for the filling of a large crack or solution cavities in rock and for grouting coarse or bouldery gravel. The proportion of sand used in the mix will depend upon the application, but care must be taken to avoid segregation of sand particles after injection. Typical cement:sand:water ratios reported are 2:(2-10):(2-5), with the proportion depending upon pump pressure, desired strength, pumpability, set time, and experience.

Admixtures are often used with cement-based grouts to improve the properties and dewatering capabilities of the mix. Pozzolans such as fly ash, pumicite, tuff and diatomite are added to increase the cementation properties and reduce segregation and shrinkage. Calcium chloride can be added to accelerate set times. Lignosulfonates are added to reduce water content and increase fluidity (pumpability). A number of other admixtures can be used to reduce segregation, increase pumpability, reduce shrinkage, or accelerate or retard setting time.

4.53.2 Clay Grout

The use of a clay mixture consisting of a dispersed clay slurry and a flocculating agent can be an effective waterproofing grout, able to penetrate finer materials than pure cement grouts. The clay grouts are very inexpensive, especially

if the clay base is locally available. A proper knowledge of the chemical and physical properties of the clay, its applications and limitations, and the proper preparation procedures is required for a successful mix (Ref. 159). Clay grouts are susceptible to degradation by groundwater flow and do not affect the soil strength appreciably. (Ref. 43).

4.53.3 Silicate Grouts

Chemical grouts have been used for treatment of fine sands and coarse silts and as a secondary treatment of rock fissures. The versatility of chemical grouts allows for closer control over the injection process, grout placement, set time, and grout strength and permeability.

Sodium silicate based solutions and gels are the most widely used and inexpensive of the chemical grouts. (Ref. 165) Mixing with a reagent in various concentrations produces a grout which can infiltrate fine-grained, cohesionless soils greater than 0.15mm (Refs. 165, 201), although certain sources quote their use in soils as fine as .02 to .08mm (Ref. 43). Soils with permeabilities as low as 10^{-3} cm/sec to 10^{-4} cm/sec have been successfully stabilized using silicate-based grouts.

The reagents used to complement the silicate grouts are usually calcium chloride (to form a high strength precipitate) or sodium bicarbonate, hydrochloric acid with copper sulfate, ethyl acetate, or sodium aluminate to form weaker, slower setting gels.

The high strength grout, i.e. the mix using calcium chloride as a reagent, is injected using a "Joosten" or "two-shot" process (Figure 26) due to instantaneous set when the solutions mix. This method is expensive and grout placement is difficult to control. An economical "one-shot" process can be used to inject the other sodium silicate-based grouts. These mixes typically have lower strengths but higher initial waterproofing capabilities than the two-shot grout. Other advantages include well-controlled set times (seconds to hours) and ease of excavation of pregrouted soils.

The waterproofing capabilities of silicate grouts depend upon the concentration of sodium silicate within the solution. Where strengthening ("consolidation") grouts may be diluted by 25-60%, waterproofing solutions are usually diluted by 70-90% (Ref. 256). These low silicate concentrations result in grouts of very low viscosity, therefore increasing pumpability and infiltration rates.

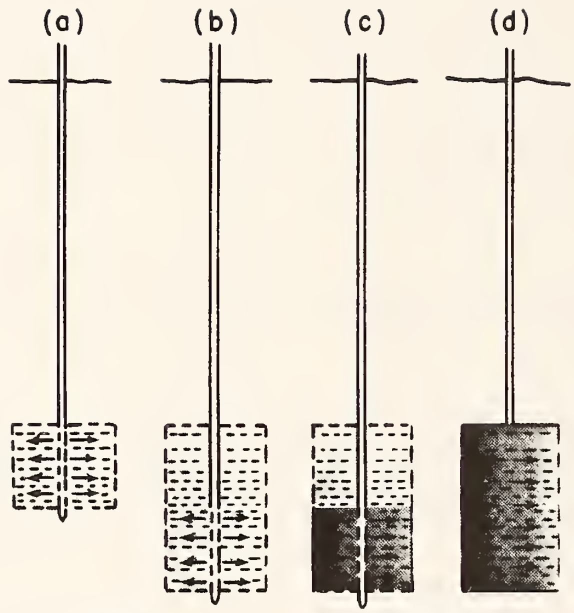


FIGURE 26 - Joosten Grouting Process (Ref. 133)

Limitations of silicate grouts for dewatering purposes are associated mainly with long-term grout degradation. The dilute solutions necessary to attain adequate waterproofing sets are susceptible to alteration by leaching and curing as water is expelled (Refs. 165, 256). This undesired phenomenon occurs mainly in grouted coarse sands and gravels or grouts which use hydrochloric acid or sodium bicarbonate as reagents.

The toxicity of silicate grouts is minimal, although direct contact with the individual components during mixture should be avoided.

4.53.4 Acrylamides

Mixtures of acrylamide and methylene-bisacrylamide can be mixed with water and a catalyst to form a versatile grout of low viscosity and controllable set time. This mixture, commonly known under its trade name AM-9, has been used to effectively stabilize soils as fine as coarse silts (d=.01-.02mm) or with permeabilities as low as 10^{-5} cm/sec. By varying the concentrations of the acrylamide and the catalyzing agents, set times are easily controlled from seconds to hours. The initial low viscosity of the solution (very near that of water) is retained by the grout for nearly 95% of its fluid life, allowing for accurate placement of the grout into the soil mass before setting. The rapid and controlled rate of setting is advantageous in properly sealing a variety of materials at any depth, even under large groundwater flows (Refs. 4, 43, 45).

Unfortunately, the great advantages offered by acrylamide grouts in terms of viscosity and set time are countered by their high costs and possible toxicity. Questions concerning this latter point have been raised in response to an apparent case of poisoning which occurred as a result of acylamide grouting in Japan (Refs. 32, 33). Despite the apparent safety of the product when caution is exercised, a recent alert by the Environmental Protection Agency (EPA) concerning a certain catalyst used in conjunction with acylamides caused sufficient controversy to prompt the manufacturer of the product to remove it from the market.

At present, acrylamide grouts nearly identical to AM-9 are manufactured in Japan and exported to a few select distributors within the United States. These grouts use a nontoxic catalyst. Very strict use regulations have been adopted by the Japanese manufacturers.

4.53.5 Lignosulfonate Grouts

Lignin is a by-product of the sulfite process in the paper industry which forms an insoluble gel when mixed with sodium dichromate. The addition of various amounts of ferric chloride results in a low viscosity grout with a controllable set time from three minutes to several hours. The resulting product is used in fine to medium sands and coarse silts with grain sizes to 0.05-0.08mm and even 0.02mm (Refs. 165, 256).

For the purpose of tunnel dewatering, relatively dilute solutions of 20-35% calcium or sodium lignosulfonate are pumped into the substrata. These dilute solutions have relatively low viscosities which can be altered to suit the characteristics of the injected medium by increasing the concentration of lignosulfonate. Reported permeabilities in fine sands grouted with a lignin based solution are as low as 10^{-9} to 10^{-10} cm/sec (Ref. 256).

Dilute lignosulfonate grouts are easily injected into even fine materials using a single-stage process. The durability of the grout under repeated cycles of wetting and drying is reported to be very good, except in very dry or wet mediums. The calcium lignosulfonate grouts appear to be more stable than the other lignin-based materials, while the sodium lignosulfonates are not recommended due to poor stability. Short term dewatering using lignosulfonates can be very effective and more economical than most other chemical grouts (Ref. 132).

The major drawback with the use of lignonsulfite grouts is the toxic properties of hexavalent chromium which is used to induce the formation of the gel. Efforts to reduce the possibility of contamination due to the hexavalent chromium have been reported (Ref. 256). A possible reduction in grout toxicity, combined with plentiful present sources of raw lignin material make the use of lignosulfite materials especially attractive for future grouting applications.

4.53.6 Resin Grouts

A variety of resins is available for use in soil and rock stabilization. In general, resin grouts provide very high strengths but are more expensive than other chemical grouts. They have been used mainly in the area of consolidating loose or unstable deposits rather than for waterproofing.

As mentioned above for other chemical grouts, poor mixing can result in leaching of toxic chemicals into groundwater. Primary applications are in fine sands and coarse silts due

to the very low initial viscosities of the resin; they are also used as secondary treatments of the areas previously grouted with coarser materials.

4.53.7 Emulsion Grouts

Emulsions are suspensions of one liquid in solution (usually water) which break up when "destabilized" after injection. The most common emulsion is composed of bitumen in colloidal solution which separates out as the stabilizing agent and is broken up or absorbed in the soil.

Bituminous emulsion grouts can be effective in sealing off flowing water if the grout coagulates properly after injection. Unfortunately, the breakdown of the grout is not reliable, and the rate of coagulation is difficult to control under variable ground conditions. Sands as fine as 0.1mm can be treated relatively economically with good long-term stability and no possibilities of toxic contamination (Ref. 256).

4.53.8 Other Grouts

Some materials available for grouting purposes are rarely used because they are expensive, have limited applications, are difficult to handle and because of limited knowledge of their properties. These materials include:

Foams - used in fine to coarse sands, they provide high strength and adequate waterproofing, but are expensive and highly toxic (Refs. 72, 165).

Bituminous asphalts - mainly utilized as a primary treatment to stop water flows. The asphalt must be heated to 400 to 600 degrees farenheit. to provide adequate pumpability.

Grouts which react with ground or groundwater chemicals - very easy to use due to the injection of a single solution, but very limited application (Ref. 256).

Vulcanizable oils - these expensive grouts are used to produce weak waterproof barriers; the initial product can be very viscous and difficult to inject, and the set time relatively long.

4.54 Grouting of Soil

Table 5 is a summary of the applications of the major grout types in soils. Initial choices may be based upon grain

TABLE 5 -- GROUT APPLICATIONS IN SOIL

SOIL TYPE	TREATMENT	REMARKS	LIMITATIONS
Coarse Sands k >10^{-1} cm/sec	Clay-bentonite grout Clay-bentonite-cement grout Cement grout and chemical grout	Penetrability a function of grain-size of soil Primary treatments often used	
Medium-Fine sands k= 10^{-1} to 10^{-3} cm/sec	Bentonite grout clay--chemical grout	Economical chemicals increase stability and accelerate set	Limited where large groundwater movements are present
	Silicate grout	One-shot process Cheapest chemical grout controllable set time Low viscosity Low permeability Two-shot process for very rapid set times	Possible long term degradation
	Lignin based grouts	Easily injected Reasonably inexpensive Controllable set time Low viscosity, permeability	Possible toxic effects
	Acrylamide (AM-9)	Controlled set time Very low viscosity Long term stability Low permeability	Very expensive Possible toxic effects Not much more effective than silicate grouts

1 cm./sec. = 0.39 in./sec.

TABLE 5 -- GROUT APPLICATIONS IN SOIL (Cont'd)

SOIL TYPE	TREATMENT	REMARKS	LIMITATIONS
	Vulcanized oils		Expensive Difficult to inject Long set time
	Asphalt emulsions	Economical Good waterproofer	Difficult to insure proper coagulation
Fine, Silty Sands, Coarse Silts k $<10^{-3}$cm/sec k $>10^{-5}$cm/sec	Dilute silicate grouts	Easily injected if diluted Inexpensive Low permeability	
	Acrylamides (AM-9)	Versatile Useable in very fine grained soils	Expensive Toxic
	Resins	Very high strengths possible	Very expensive

1 cm./sec. = 0.39 in./sec.

size or permeability, but the final choice may depend upon local experience, grout availability, costs, or a special consideration such as flowing water.

Cost considerations for the various grout types must include the costs of the material as well as injection costs. Table 6 presents the relative costs of grout materials using cement grout as a basis. The versatility of most chemical grouts is countered by their cost. This expense is offset to some degree by the fact that particulate grouts may be three to five times more costly to pump into the ground than the less viscous chemical solutions (Ref. 201), and the volume of chemical grout required to stabilize a volume of soil may be considerably less than if particulate grout were used.

The pressure used to inject grout into soil varies with the grout hole depth, grout type, and method of injection. Usually, the maximum allowable grout pressure is limited to the effective overburden stress, although some sources cite the use of pressures in excess of three or four times the overburden stress (Ref. 201). This may be due to substantial pressure losses in the piping at large depths, especially for more viscous grouts. As a project progresses, either a trial-and-error procedure or a test section may be used to determine the maximum allowable grout pressures for a given site.

In general, coarse sands and gravels are initially grouted using clay or clay-cement grouts because they are inexpensive. Problems associated with pumpability, set time, and segregation are dealt with through the addition of chemical additives.

A rule of thumb generally employed in the grouting of coarse soils with suspension grouts uses a ratio of effective grain size of the particles within the grout to soil particles within the stratum. The "groutability ratio" is thus defined as

$$\frac{D15}{D_{85}}$$

where D_{15} = diameter of grains in stratum where 15% of the soil mass is finer

and D_{85} = diameter of particles within the grout where 85% of the particles are finer

Various sources state that this ratio should be at least 19, and preferably greater than 24, to ensure adequate penetration of grout into soil voids.

TABLE 6 -- RELATIVE COSTS OF GROUT MATERIALS (Refs. 128, 185, 237, 253)

GROUTING MATERIAL	RELATIVE COST FOR MATERIALS			RELATIVE COST IN PLACE		
	U.K.	FRANCE	U.S.	U.K.	FRANCE	U.S.
Cement-Bentonite	1		-- 1		--	1
Deflocculated Bentonite	1.8		-- 1.8	1	--	1
Cement	--		1.0 4.2		1	
Silicates						
-one-shot, dilute	3.3-7	2-4	1.3 --	1.2 to 14	1-2.6	1.4-2.7
-one-shot, concentrated		6	2.9 --			
-two-shot	6	--	-- 10.7		--	
Lignosulfates	--	--	1.65 6.5-8		1.3-2.6	
Acrylamides	11-27	--	7.0 50-130		1.3-2.6	
Resins	--	10-500	6.0 10-40 250-500		--	
Bituminous Emulsion	--	--	-- 6-12	--	--	--

Secondary treatment of coarse soils and primary treatment of fine to medium sands are typically accomplished with inexpensive silicate based grouts. This approach is used frequently in Europe and Great Britain (Refs. 71, 150, 197, 209). Other common chemical grouts used in these types of soil are the more expensive lignin-based materials.

Finer grained soils can be effectively penetrated only with the low viscosity chemical resins such as the acrylamide AM-9 or other organic resins. These grouts can be very effective waterproofing agents, but are typically expensive (Refs. 7, 46).

4.55 Grouting of Rock

The treatment of highly fractured or weathered rock may pose problems if the openings are of variable width and filled with weathered material. The location and treatment of large fissures and the cleaning of filler materials are aspects of rock grouting which must be considered.

Particulate grouts are typically the most economical means of reducing groundwater flows in rock. Large openings can be filled with coarse sand-cement grouts, while narrower fissures can be effectively treated with clay-cement, clay, or even chemical grouts. Packer grouting or tube-a-manchette techniques are especially suited for treating individual rock features with special grout mixes. A generally accepted rule of thumb states that the dimension of the crack width should be at least three times the diameter of the largest particles in the grout. Table 7 is a summary of grouting applications in rock.

After drilling, grout holes are flushed to remove cuttings and to clean out filler material, and are then pressure tested to determine rock permeability for proper selection of grout consistency. When grouting from within the tunnel, it is possible to obtain comparable information on rock conditions by monitoring the rate of seepage into the grout hole (Ref. 226).

Grouting pressures used in rock may be larger than the overburden stress, especially if the overlying rock is sound. A conservative commonly used rule of thumb for grout pressures is one pound per square inch per foot of depth (Ref. 260). More detailed guidelines are presented in Figure 27. An observational approach is recommended for determining appropriate grout pressures for a site; commonly used indicators of excessive pressures are large grout flows, leakage of grout to the surface or surface heave (Ref. 201).

TABLE 7 -- GROUT APPLICATIONS IN ROCK

	TREATMENT	REMARKS	LIMITATIONS
Rock Fissures -large	Sand-cement grout Cement grout with fillers Hot asphalt Clay or bentonite-cement grout	Usually economical Penetration depends on grain size of grout components Admixtures adjust set time, flowability, stability	Possible segregation Limited where there are groundwater flows (except stability)
-small	Clay-bentonite cement grout Clay or bentonite grout Cement grout and chemical grout	Economical Admixtures adjust set time, flowability, stability Primary and secondary treatments often used	Possible segregation

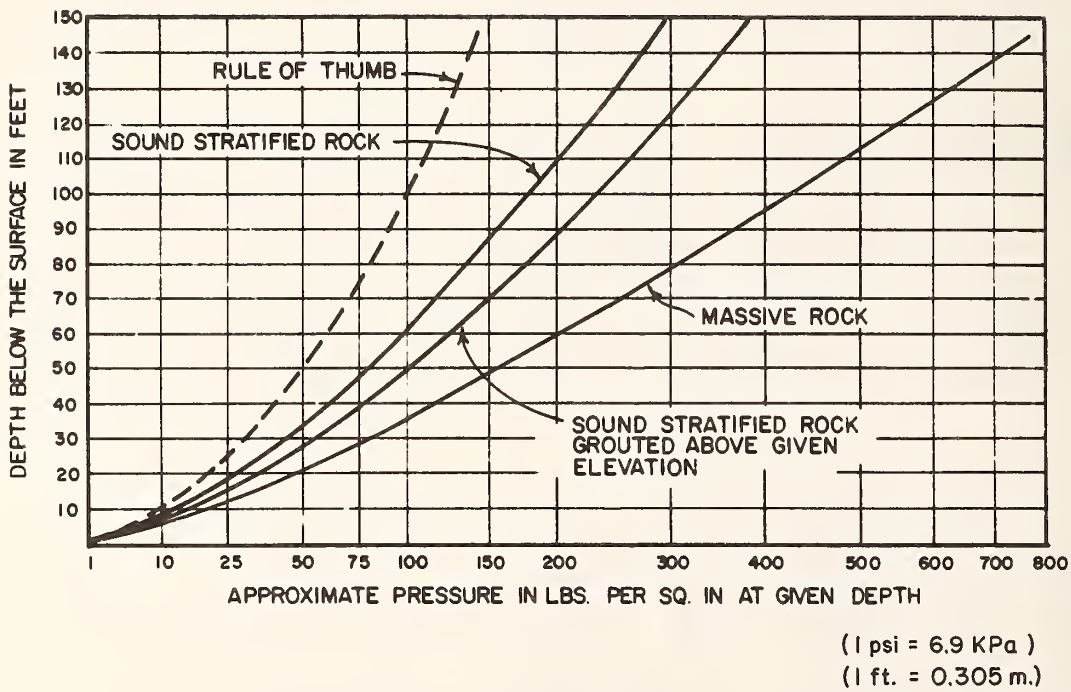


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FIGURE 27 - Guidelines for Grouting Pressure in Rock (Ref. 81)

Termination of rock grouting is signaled by indicators that the grouted zone is sufficiently impervious. As with soil grouting, this may be indicated by reduced water flows, difficult pumping of grout, surface heave, or pressure testing of the grouted zone.

4.60 FREEZING

Ground freezing is simply the cooling of the subsurface so that pore water is converted into ice, imparting strength and impermeability to the ground formations. Although its use is reported as far back as 1862, the method has been used sparingly in civil engineering because of its high cost and because the results are temporary. As such, the process of ground freezing has frequently been applied only in "emergency" situations, and then only by a limited number of specialist contractors. Recent trends in construction are to consider freezing as a viable construction alternative due mainly to the development of otherwise unsuitable or difficult sites and to the increased cost competitiveness of freezing with other construction methods.

Ground freezing is typically accomplished through the removal of heat from the ground via probes inserted at intervals. The probes consist of 2-inch (5.1cm) to 6-inch (15.2cm) diameter steel or copper pipes into which are placed 0.75-inch (1.9cm) to 3-inch (7.6cm) inner pipes; the coolant is pumped down through the inner pipe and returns in the annular space between the two. The coolant commonly used is a calcium chloride brine. As the brine rises in the outer pipe, it removes heat from the surrounding soil and increases in temperature. At the surface, the brine is then recirculated through a refrigerator plant, is recooled, and reintroduced into the ground. As the temperature of the ground decreases, the water within the voids begins to freeze, "cementing" the soil particles together. The colder the brine coolant, the quicker the groundwater will freeze. Brine can typically be cooled to from -20 degrees to -40 degrees centigrade (C), imparting an average temperature in the frozen soil of -5 degrees C to -15 degrees C. A typical brine freezing system is depicted in Figure 28.

Other cooling systems are available for situations where rapid freezing and lower temperatures are required. Liquid nitrogen provides coolant temperatures of -196 degrees C but cannot be reused and thus requires a continual supply of coolant. Freezing by this method is therefore very costly and is used primarily in emergency situations where rapid freezing is necessary. A third coolant process employs a closed system and a circulated liquid/gas coolant such as freon; this type of system was used

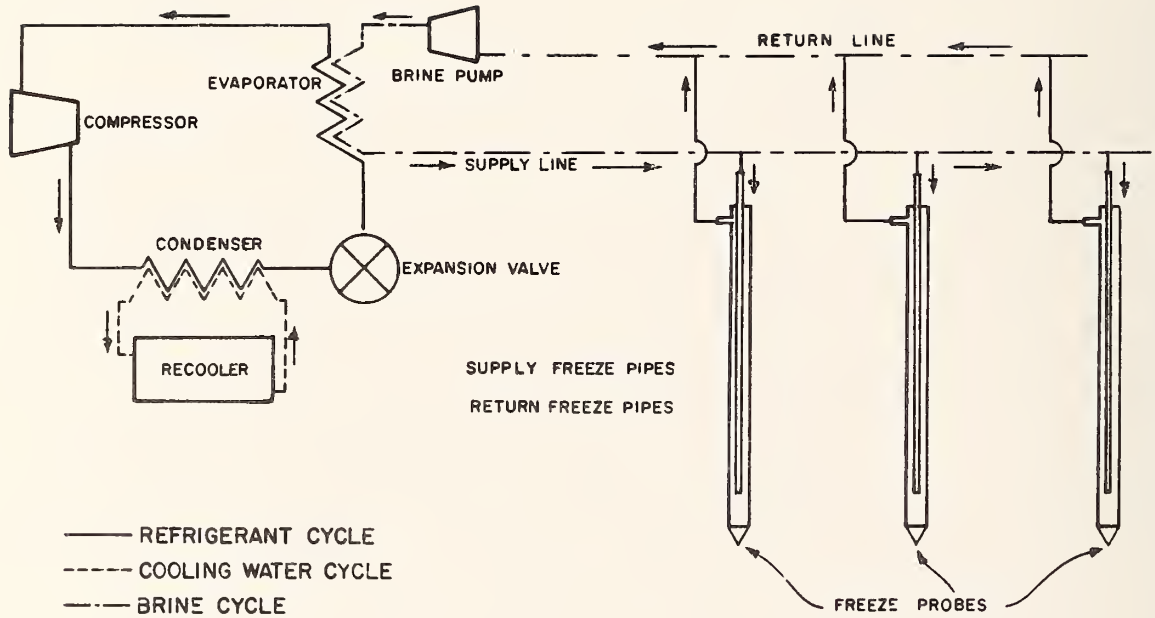


FIGURE 28 - Typical Brine Freezing System (Ref. 59)

in construction of the Helsinki Metro, Finland (Ref. 201). At times a dual system, using liquid nitrogen for the initial freeze and brine for maintaining the freeze, is employed where both rapid freezing and long maintenance periods are required.

The mechanics of ground freezing have been studied in detail with respect to the time and energy requirements needed to freeze a soil mass and the properties of the resulting product. Virtually any type of soil or rock can be stabilized by freezing if moisture is present within the pore spaces (Refs. 108, 152, 201). It can be used to stabilize fine sands and silts, as well as to form a barrier to flow in coarse sands and gravels or fissured rock. The pattern of soil freezing typically develops radially from each inserted probe until the frozen zones from adjacent probes overlap. Basic parameters which govern the efficiency of the freezing system include the initial temperature of the soil mass, the heat capacity of the soil and pore fluid, the latent heat of fusion of the pore liquid, and the thermal conductivity of the soil mass. It has been shown that the latent heat of fusion, i.e., the amount of heat required to convert the pore water to ice, is the most important factor in estimating energy requirements. This parameter is, in turn, dependent upon the dry unit weight and water content of the soil mass. Once the required volume of soil has been frozen, the energy required to maintain the frozen section diminishes markedly since the only sources of external heat occur at the perimeter of the frozen zone.

Typical application of the freezing technique to a tunnel dewatering problem is depicted in Figures 29 (a) and (b). In case (a), the ground around the sides and crown of the proposed tunnel is effectively frozen down from the ground surface; the probes along the tunnel sides are usually driven to an impermeable stratum below the tunnel invert. It is an advantage to have the frozen zone along the sides penetrate into the impervious stratum in order to withstand the high gradients which can develop at the bottom of a partially penetrating cutoff. Typical spacings of the freeze probes in this situation are 3 to 5 feet (0.9-1.5m). Often temperature monitors, such as thermistors or thermocouples, are installed at critical locations along the tunnel alignment to monitor the subsurface temperature and thus the effectiveness of the system. Where it is not possible to seal off the flow of water by freezing into an underlying impervious stratum, freezing may be accomplished by horizontal freeze pipes around the perimeter of the tunnel as shown in Figure 29 (b). This approach eliminates the need for surface access above the tunnel, but requires additional space within the tunnel itself for installation of the freeze probes. This approach has been used for

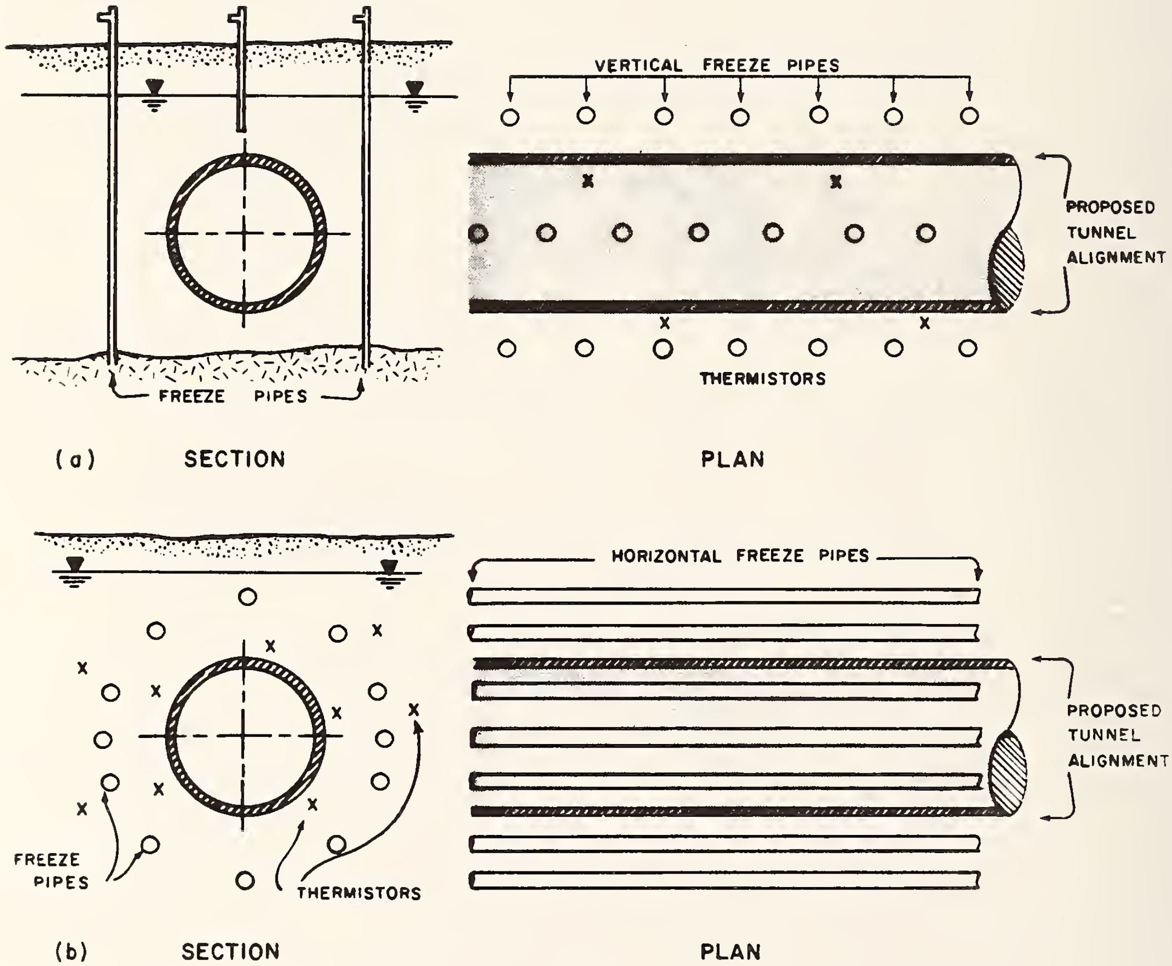


FIGURE 29 - Schematics of Alternate Systems of Freezing from Surface and Tunnel

the construction of tunnels in Helsinki, Finland; Chambery, France (Ref. 201); New York, N.Y.; and Washington, D.C. (Ref. 152). Again, a temperature monitoring system is usually installed to ensure adequate stabilization within the frozen annulus around the tunnel perimeter.

The economics of freezing as a construction alternative depends upon the length of time required to attain complete freezing of the desired zone and the number and spacing of freeze probes. Figure 30 illustrates the importance of coolant type and probe spacing upon the time required to freeze a given volume of soil. Requirements pertaining to the desired thickness of the frozen zone, the number of probes and the amount of time available must be considered in conjunction with one another to arrive at the most economical, effective solution. These parameters depend, in turn, upon site-dependent variables such as soil type, permeability, groundwater conditions, pore fluid chemicals, adjacent structures and utilities, etc. For example, to effectively freeze a 2.5-foot (0.76m) diameter zone of soil using 3 inch (7.6cm) diameter freeze pipes through which a calcium chloride brine is being pumped would require between 100 and 215 hours. The presence of local heat sources (buildings, utilities, etc.) or flowing groundwater could significantly increase the time required to achieve a frozen zone, however.

Although the freezing method has been practiced for over a hundred years, its use has been limited mostly to shafts and open cut excavations until just recently. The first tunneling project in which freezing was used as one of the primary groundwater control methods was on a sewer tunnel project on the west side of Manhattan (Refs. 101, 102) (Figure 31). Due to the nature of the soils and concern over possible settlement of a nearby railroad, dewatering was not allowed. Compressed air was used in the soft ground portion with the rock portion to be constructed in free air. The access shaft was located in a transition zone between soil and rock. It was originally designed to be constructed under compressed air; however, local citizen pressure caused the city to revise the contract to disallow any blasting after 10:00 pm. Maintaining the air pressure overnight without working was very expensive, and the City, therefore, agreed to a changed condition for construction and freezing was permitted for groundwater control in this mixed face area. A system was designed using a calcium chloride brine as the secondary coolant. The brine was circulated through refrigeration plants which cooled the brine to a temperature of -21 degrees F (-29 degrees C). Approximately 350 feet (106.7m) was frozen to the south of the access shaft in three segments of roughly 120 feet (36.6m) each. One segment of roughly 100 feet (30.5m) was frozen to the north of the shaft. A crossover row of freeze pipes was boxed in the end of each segment in an attempt to form closure of the wall. However, two episodes of cave-in occurred with the soil

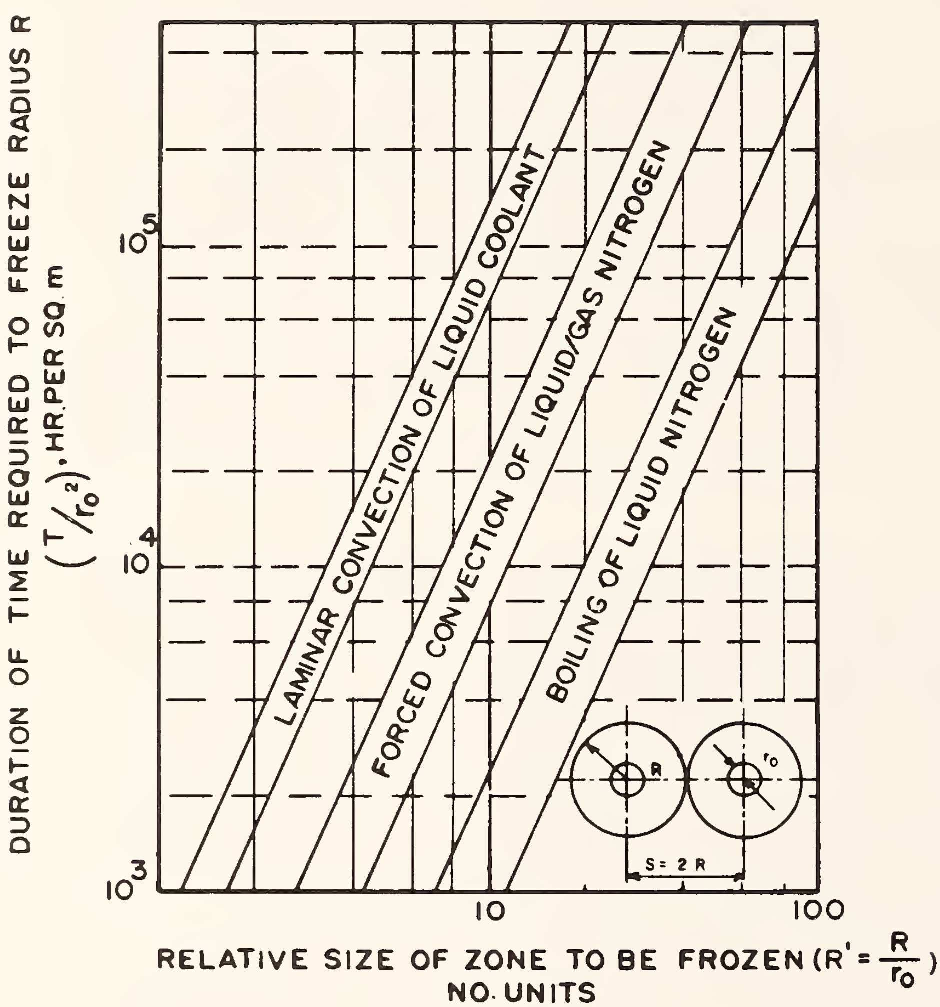


FIGURE 30 - Generalized Relationship Between Freeze Pipe Size, Spacing, and Time (Ref. 152)

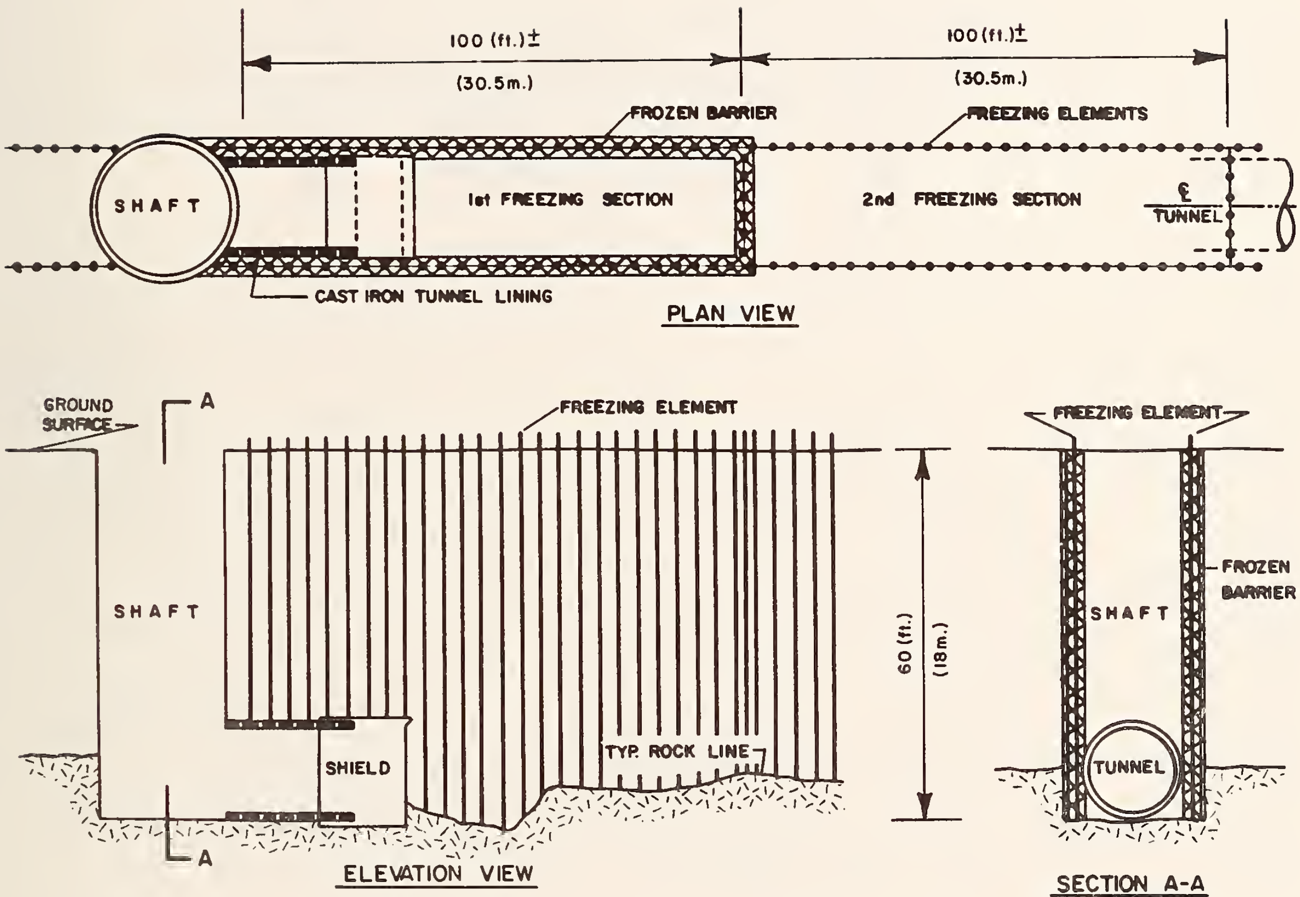


FIGURE 31 - Ground Freezing for Sewer Tunnel, New York City (Ref. 101)

over the arch between the frozen walls. The cave-ins were backfilled and refrozen over an extended period. A section of the rock tunnel section was also frozen because the rock dipped below the top of the tunnel. Problems with cave-ins occurred due to inadequate closure of the freeze wall. The problems are believed to have resulted from a pervious zone through which water flowed across the section. More recent experience indicates that such problems can be avoided by careful monitoring of pipe location and groundwater gradients from which average seepage velocities can be calculated.

When the tunnel was completed through the transition zone, the air was turned on in the south heading. An air pressure of over 12 psi was required to keep the tunnel dry. However, this pressure caused heaving in the street. When the pressure was under 12 psi, water entered the tunnel. Therefore, a dewatering system of ejectors was tested and used to reduce the required air pressure.

Another example of using the brine freezing technique was for the drilling of twin rapid transit tunnels under the Tokyo River (Ref. 31). The purpose was to stabilize weak riverbed silts (Figure 32). In this case, access shafts on either side of the river, 156 feet (47.5m) apart were used for the installation of 200 horizontal freeze pipes. The soil was frozen in a rectangular shaped fashion, 132 feet (40.2m) wide by 56 feet (17.1m) deep, under 20 feet (6.1m) of cover. Control of the freeze was accomplished using various techniques. The conventional brine system was used. Areas within the rectangular box were allowed to drain by the use of 30, 4-inch (120mm) diameter strainer pipes under vacuum. The riverbed was insulated from being a heat source by dredging and installing panels of foam plastic sheets sandwiched between steel plates. These sheets were 50 feet (15.2m) long by 13 feet (4.0m) wide and were bolted onto H-beams placed on a sand mat. Freezing pipes also ran through the panels. To protect against possible heave near an expressway pier, a wall of 16- to 24-inch, (406-610mm) pipes was driven horizontally at the edge of the freeze zone to prevent the spread of freezing. Also, hot water pipes were installed vertically along the face of the pier to prevent freezing.

The major advantages credited to ground freezing as a method for preventing water seepage into tunnels are the wide range of soil types to which it is applicable and the strength imparted to the frozen soil. The freezing technique itself is not subject to variations in soil permeabilities as are other dewatering or stabilization methods (such as pumping or grouting), and so a continuous stable zone is probable. Monitoring of the frozen zone is possible using simple thermometers inserted at critical locations. Where time constraints are important, a liquid nitrogen system can provide a quick frozen section.

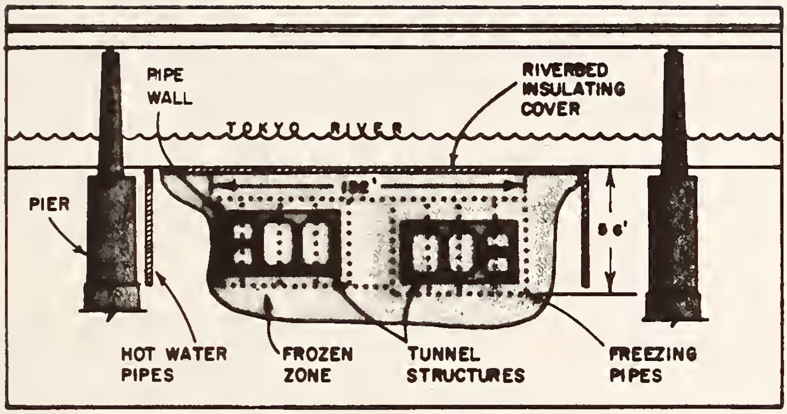


FIGURE 32 - Ground Freezing Under Tokyo
River (Ref. 31)

This type of freezing can economically compete with other stabilization methods on jobs which require maintainance of the frozen section for relatively short durations.

The main disadvantage commonly associated with ground freezing is its expense. The high cost of setting up and maintaining a refrigerant plant at the surface for brine and liquid/gas systems accumulates with the length of the required freeze; liquid nitrogen systems, which need a continual source of nitrogen, are also only economical in very short-term applications. In both cases, the result is only temporary and cannot aid in keeping water out of the tunnel after construction is completed. System leaks due to faulty tubing or unusually large heat sources (such as moving water or sewage in adjacent utilities) can increase the cost of maintaining the frozen zone.

Excessive groundwater flow may make freezing impractical because if heat returns to the soil by flowing groundwater faster than it can be removed by the coolant system, the ground cannot be frozen. Therefore, a knowledge of groundwater gradients and soil permeability is essential to the design of the freezing system. A generally accepted rule of thumb states that groundwater flows in excess of 3 to 6 feet/day (1 to 2 m/day) will make the closing of the frozen zone impractical or impossible. Under high groundwater gradients the frozen zone may never fully close, leaving "windows" which will lead to the eventual deterioration of the frozen wall. If the freezing pipes do not penetrate an impervious stratum to form a closure, or are not properly aligned when installed, the time required to attain the proper stabilization may be significantly increased. Soil to a depth of approximately 0.4 times the wall thickness will freeze below the pipes. To insure proper closure between adjacent pipes, good vertical alignment is desirable.

Water which is contained within the frozen area usually does not pose a problem since it is cut off from any recharge. However, attempting to pump out the inside before or immediately after closure can cause destruction of the wall. Besides larger groundwater flows and improper pipe alignment, another consideration for freeze wall stability involves protection from local sources of heat such as sewer lines, steam lines, etc. These external heat sources resist the cooling action of the refrigerant by continually resupplying heat to the ground. The frozen zone can be protected by isolating the heat source and installing freeze pipes at closer spacings within the area of concern. Care should also be taken to avoid freezing of local utility lines which lie too close to the frozen zone.

One of the more important aspects of ground freezing in urban tunneling is its effect on the soil structure itself, especially in terms of strength, possible heave, and subsequent thaw and settlement. A number of sources quote the factors affecting ultimate strength and creep characteristics of the frozen soil mass (Refs. 108, 231).

In the dewatering of tunnels, these strength parameters become important when the frozen zone is used as a structural arch or method of underpinning in addition to waterproofing. In general, the strength of the frozen granular soil increases with decreasing temperature and increasing water content. Clays and silts do not generally show the strength increase at higher water contents that sands do. Creep deformation is generally not a problem in temporary tunnel excavations and may be countered by decreasing coolant temperatures.

Heave may constitute a problem in freezing beneath urban areas due to cracking of overlying pavements or structures. The phenomenon of ground heave occurs predominantely in finer grained soils, such as silts or clayey silts, due to: (1) capillary migration of pore water which forms an ice lens, and (2) the volumetric expansion of pore water as it changes from the liquid to the solid phase. Ice lenses will generally form in a direction perpendicular to the direction of heat flow. Volumetric expansion of water due to freezing is about 9%, so the total volumetric expansion of the soil mass may be 3% to 4% depending upon void ratio (Ref. 92). The actual upward expansion of the soil mass may therefore total only 1% to 1.5% and must overcome overburden pressures before affecting surface features. Free draining cohesionless soils will generally not exhibit heave because the pressures developed by the soil-ice system can be relieved by the movement of water through the voids. Heave in finer grained soils of low permeability can be limited by rapid freezing to eliminate the formation of segregated ice lenses. In general, surface heave due to subsurface freezing will amount to less than a few inches (Ref. 231) but may still require monitoring in urban areas.

Since ground freezing is only a temporary method of ground support and groundwater control, provisions must be made to deal with post-treatment ground control. Of particular interest in tunneling is the exclusion of water from the completed tunnel and subsequent movements between the tunnel and thawed ground mass. Water exclusion is typically accomplished by grouting behind the tunnel liner during the freezing stage. But the formation of a zone of frost along the soil face may result in a void after the soil has thawed. Movements into this void can be limited by careful two-stage grouting as the ground mass returns to its ambient temperature (Ref. 201).

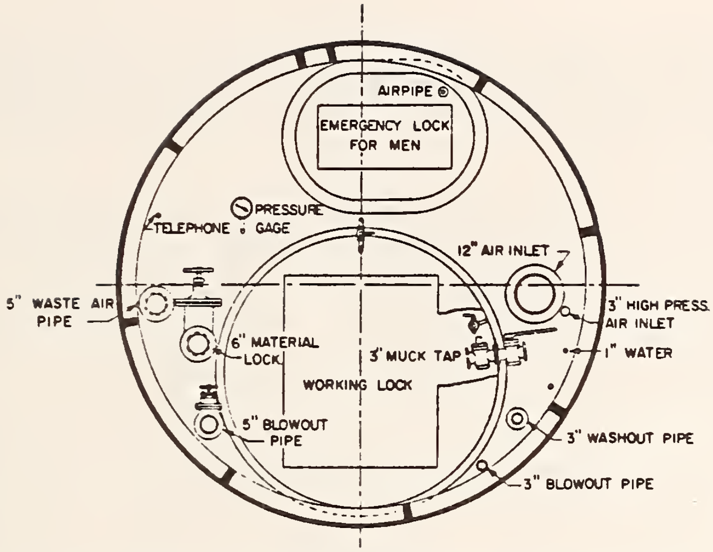
Settlements due to ground thawing may occur in finer grained soils due to reduced compressibilities and shear strengths after freezing. They may also occur due to insufficient contact between tunnel liner and adjacent soil as mentioned above. One source cites settlement at a tunnel in Chambery, France, of nearly four times the heave measured during freezing (Ref. 201), but this is a very unusual occurrence. Generally, the amount of settlement on thawing is less than the uplift during freezing.

The cost of freeze systems is usually not competitive with other methods of groundwater control unless other factors are considered, such as freezing in lieu of underpinning or an extremely difficult groundwater or soil stability problem that must be handled. The method is advantageous in questionable situations due to the confidence which can be developed concerning the strength and impermeability of a properly frozen soil mass. Since these properties are imparted to the soil before excavation, a properly frozen soil mass requires no further treatment during the construction phase. Thus, costs and construction schedules are generally more precise than with other methods for controlling difficult groundwater conditions. Although the method is still practiced largely by specialty contractors in special situations, advances in technology and experience in field applications are making the method more feasible for conventional projects. Freezing for tunnel construction under difficult groundwater conditions is especially useful in sensitive urban environments.

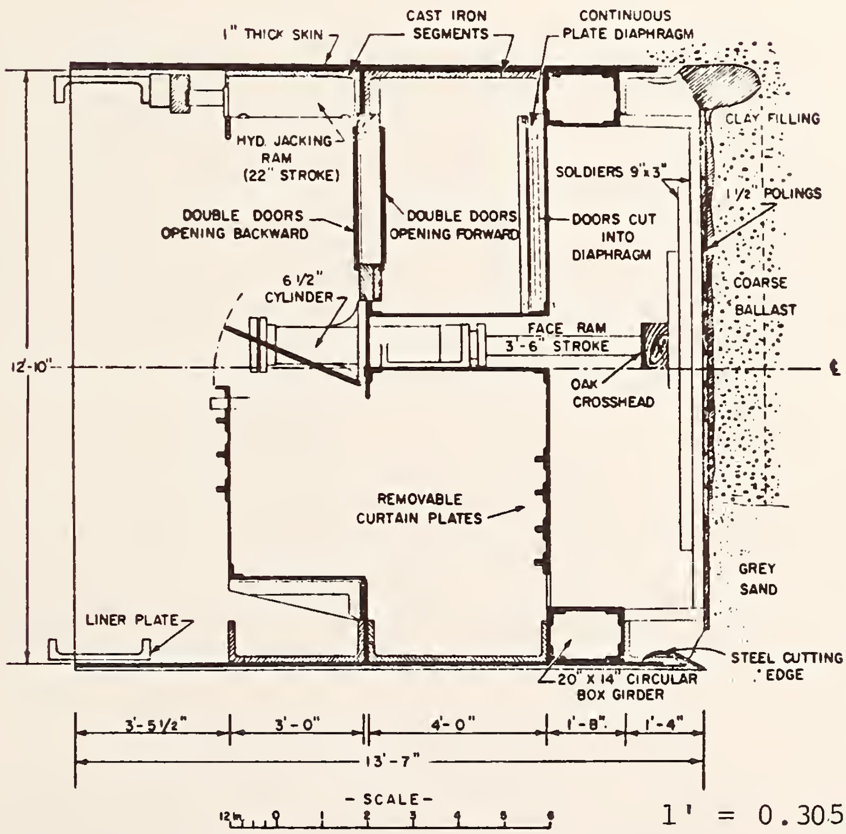
4.70 COMPRESSED AIR

Compressed air has been used as an aid in tunnel construction for over a hundred years. The first development and patent of the concept of compressed air was made by Thomas Cochrane in 1830 in England. The process was thereafter used intermittently for the sinking of caissons and construction of bridge piers below water level. The first use in actual tunneling occurred in 1879 concurrently in Antwerp, Belgium and beneath the Hudson River in New York. Since that time, compressed air has been utilized on innumerable tunneling projects through difficult water bearing strata.

The compressed air process simply involves the introduction of pressurized air into an airtight compartment to support saturated soil. The compartment typically lies at the heading of the tunnel inside a prefabricated shield or tunnel liner and is separated by an air lock or decompression chamber (see Figure 33). The basic design of air locks has changed little since their introduction many years ago. The lock illustrated in Figure 33 was used in 1899. The air pressure required depends upon the water pressure which exists at the tunnel face, the



a) BULKHEAD & AIRLOCKS; OUTSIDE ELEVATION



1' = 0.305m.

b) SECTION OF SHIELD SHOWING MAJOR COMPONENTS

FIGURE 33 - Compressed Air Shield, Greenwich Tunnel, London (Ref. 78)

type of material being supported and the "cover" available above the tunnel crown. If properly designed, a system using compressed air can be very effective in stabilizing running silts and fine sands below the water table.

The major factors which govern the use of compressed air in tunneling are: (1) the necessity for large capacity compressors and air reservoirs; (2) the ability to apply reasonable working pressures to control water and prevent large air losses or "blow-outs"; and (3) the necessity for carefully monitored working conditions and decompression chambers for workers.

Large volumes of pressurized air are required to compensate for losses through the liner and tunnel face. The interface of air and water is not stable, but rather dynamic, with air continually being diffused and lost in percolation through the water. This loss, which occurs predominantly in sands and gravels with high permeabilities, must be continually resupplied. To minimize losses along the tunnel length, the liner should be continuously sealed to prevent air loss to the adjacent soils. Sealing may often be accomplished through grouting behind liner plates or sealing of joints with mud, grout or gaskets.

Since the water head present at the tunnel face varies by an amount equal to the tunnel diameter, an applied internal air pressure must be large enough to prevent inflow at the tunnel invert, but small enough to preclude blowout at the crown, as shown in Figure 34. Usually, the pressure head at the springline is the design point with a rule of thumb of 1/2 psi (3.5×10^{-3} Pa) of air pressure per foot head of water (0.3m, or 1.1×10^{-2} Pa/meter). If the pressure at the crown is too high, streams of air may migrate through the overlying water to the surface, creating large losses of air pressure and subsequent instability at the face. Early dealings with compressed air at relatively high pressures resulted in just such blowouts which endangered many lives. Problems occurred in the construction of the Hudson River tunnels (in which twenty lives were lost), the Glasgow District Railway (Scotland), and the Blackwall Tunnel (London) (Ref. 78). Usually, the pressure calculated using the above rule of thumb may be reduced by 10% to 30% to account for frictional resistance of the flowing water through the soil mass (Ref. 178). In situations where there is little available cover to prevent large air losses, the interior pressure may be reduced to a point which allows small flows at the tunnel invert, which are then removed from the tunnel by sump pumps.

A major drawback to working under compressed air is the decompression time and shortened work shifts which accompany higher working pressures. Records of early tunnel driving under compressed air give numerous accounts of "compressed air sickness" or "the bends" which afflicted workers laboring under compressed

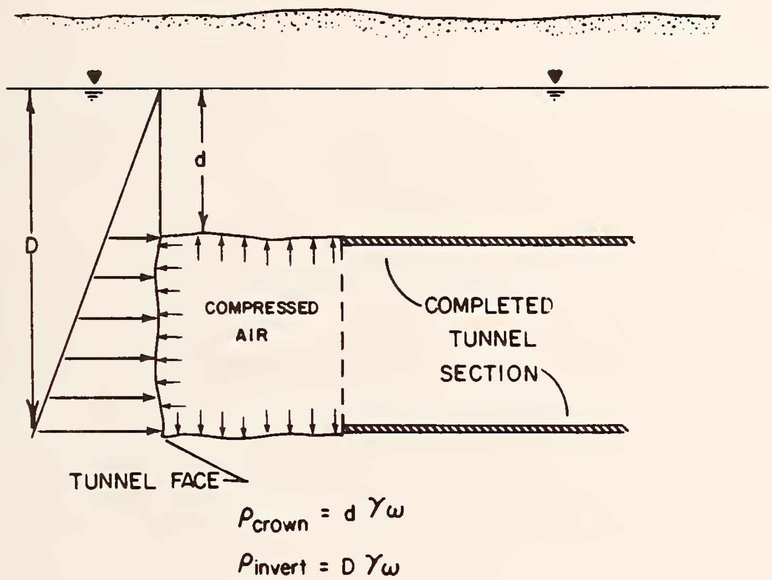


Figure 34 - Compressed Air Principle of Operation

air. Caused by rapid decompression times, the symptoms resulted from bubbles of nitrogen gas which came out of solution in the bloodstream as the pressure decreased. Construction of the Pennsylvania Railroad East River Tunnels resulted in nearly 3700 cases of compressed air sickness (Ref. 136). Although many cases are easily treated by recompression to the working pressure, severe injuries and death may occur, especially in situations where workers spent long periods of time under large pressures--up to 40 to 50 psi (276 to 345 x 10^{-3}Pa).

Present day compressed air tunneling requires adherence to stringent safety measures concerning allowable working pressures, maximum work shift, decompression facilities and medical and emergency capabilities. Table 8 illustrates guidelines established by OSHA for working under compressed air. Typically, a person is allowed to work an 8-hour shift with only 3 minutes of decompression time when working in pressures up to 12 psi (83 x 10^{-3}Pa). Over 12 psi, the allowable shift time decreases and decompression time increases. For example, at 20 psi (138 x 10^{-3}Pa), either a 4-hour work shift requiring 43 minutes for decompression, or a 3-hour work shift requiring 15 minutes is permitted. One work shift is allowed per 24-hour period. It is, therefore, obvious that the cost for labor greatly increases when operating pressures over 12 psi are required.

If conditions permit, the contractor or engineer can save on the cost of labor if he can be sure that there is no need for pressures in excess of 12 psi (83 x 10^{-3}Pa). This can sometimes be accomplished by utilizing a limited dewatering scheme as shown in Figure 35. The wells or well points lower the water level until the pressure required within the tunnel is below 12 psi. In shallow tunnels, predrainage may be used to reduce the required air pressures, not only for labor considerations, but also to prevent air losses, heave, or blowouts at the tunnel face.

Compressed air was used in sections of the BART system in San Francisco to protect against settlements in layers of peat and bay mud. The subway tunnels between Powell Street Station and Civic Center Station required a combination of dewatering, compressed air and grouting. Dewatering was permitted for the first 500 feet (152m) of a 2,100-foot (335m), 4-tunnel run, but compressed air was specified for the remaining 600 feet (183m) and for the connection into the existing station excavation. The shield used was specially designed together with muck cars to handle 5-foot (1.5m) shoves. Because of the high cost of this setup, the shield has to be retrieved and used four times. Ejectors were used because close spacing was needed to completely drain the various sand strata. Once the locks were set, widely spaced wells were used to enable the tunnel to be driven in reduced air up to the limit of the area

TABLE 8 -- SELECTED HOURS OF LABOR AND DECOMPRESSION TIMES

U. S. DEPARTMENT OF LABOR

SAFETY AND HEALTH REGULATIONS FOR CONSTRUCTION

DATED JUNE 24, 1974

Pressure psi	Pressure kPa	Hours of Labor	Decompression Time: Min.
0-12	0-83	8	3
16	110	6	33
		4	7
20	138	4	43
		3	15
24	166	4	92*
		3	52
28	193	3	98*
		2	41
32	221	2	85*
		1-1/2	43

*Decompression times in excess of 75 minutes require a special decompression facility

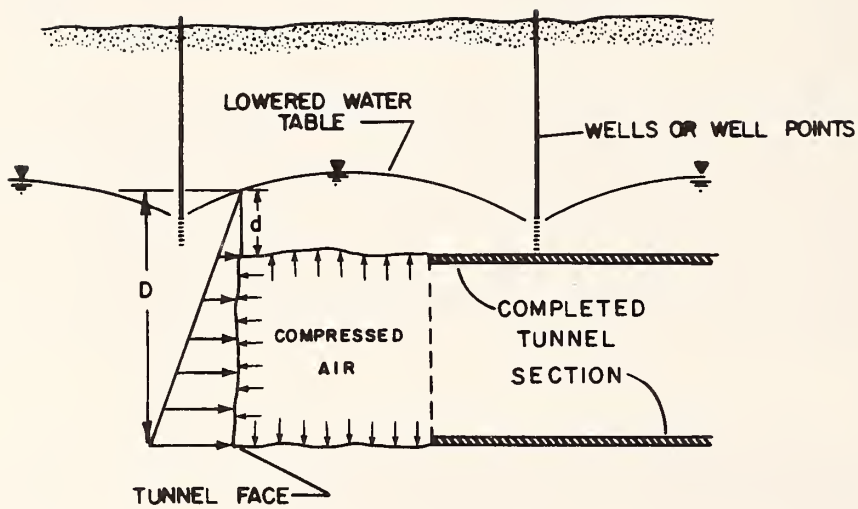


FIGURE 35 - Compressed Air in Combination with Dewatering

where dewatering was allowed. The remainder had to be driven in compressed air alone due to the existence of a compressible peat layer in the area around Civic Center Station. The station itself was constructed in open cut with the aid of a tremied concrete cutoff wall. Recharge was used to maintain piezometric levels outside the cutoff walls which were used for dewatering of mezzanines, as well as for dewatering and pressure relief of the station. The problem involved bringing a compressed air tunnel into an existing dewatered station excavation. This was accomplished by first chemically grouting the transition zone and then by driving steel bars from the station out around the tunnel. The remainder of the tunnel was hand-mined with lagging boards placed behind the steel bars for support, so that the shield could then be removed. The above situation is an excellent example of the use of combined groundwater control methods to solve an unusual problem.

Compressed air may be used in many instances where a possibility of settlement exists, such as in urban areas. It also has the advantage that, since it is applied directly within the tunnel, it has a built-in way of solving access problems. For example, no relocation of utilities is required, tunneling under surface water bodies can be accomplished, and the depth of the tunnel is not as significant to the cost of groundwater control (unless the required pressure is too high for safety reasons).

4.80 SLURRY SHIELDS AND EARTH PRESSURE BALANCE SHIELDS

Slurry shields and earth pressure balance shields are tunneling devices designed for the driving of soft ground tunnels. They are tunneling systems developed for use in lieu of compressed air and under conditions of limited cover where compressed air methods are not feasible. The shields are designed to permit tunnel crews to work in free air while giving support to the face. Slurry shields achieve this by means of a slurry-filled chamber located behind the cutter wheel. Earth pressure balance shields do it by means of excavated soil stored in a sealed chamber behind the cutter wheel. Pressure on the face is regulated by controlling the volume of excavated soil or slurry removed from the storage compartment. Schematic representations of a slurry shield and an earth balance shield are presented in Figures 36 and 37.

While these specialized shields do effectively control groundwater inflows, their primary benefits are control of ground movements and maintenance of face stability. The machines are best suited for use in loose to medium-dense saturated sands, silty sands, and soft silty clays. Grain size distribution suitable to slurry and earth pressure balance shield application is illustrated in Figure 38.

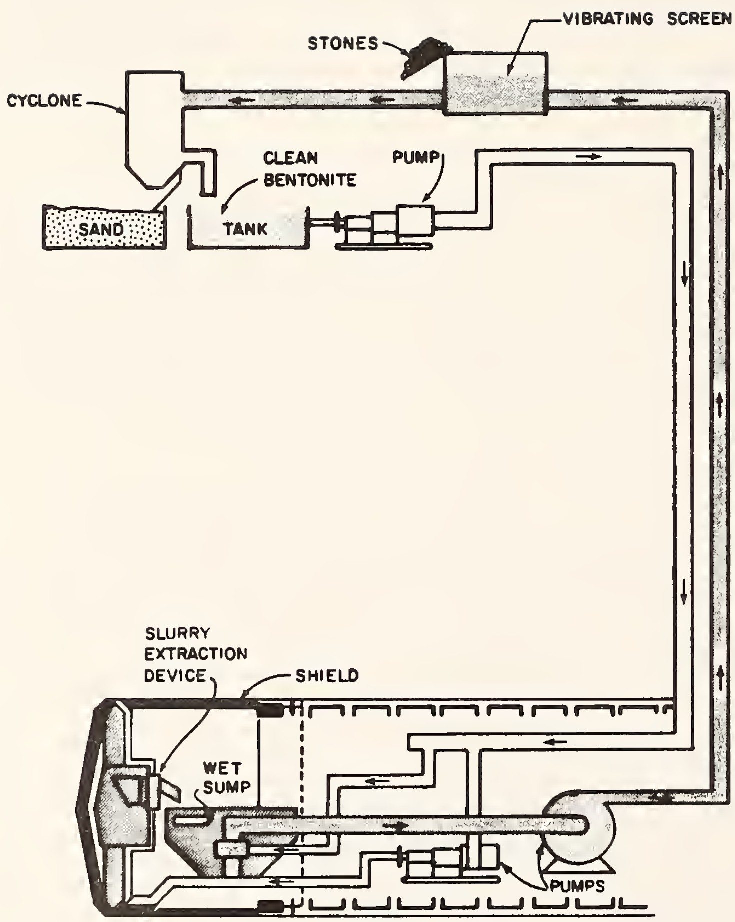
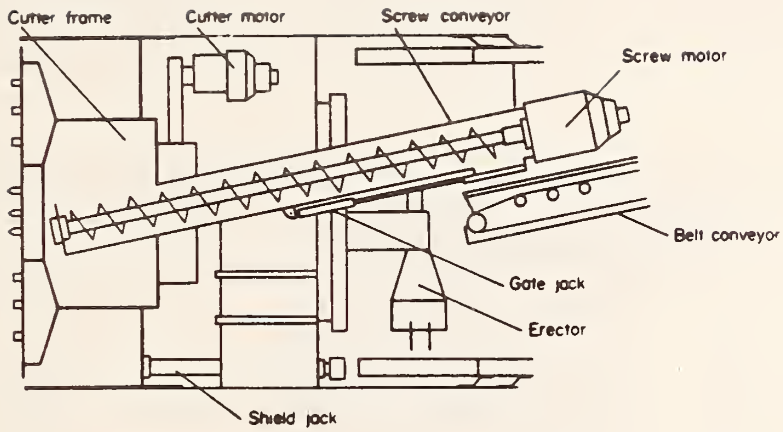
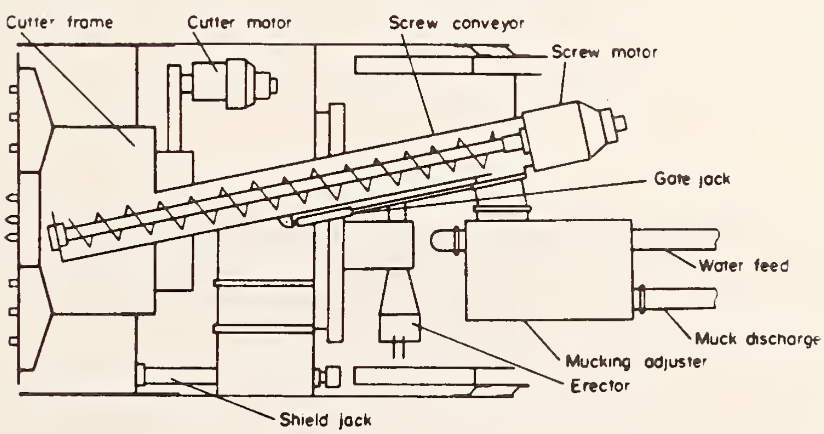


FIGURE 36 - Schematic of Slurry Shield (Ref. 201)



a) SOIL PRESSURE TYPE



b) WATER PRESSURE TYPE

FIGURE 37 - Schematics of Earth Pressure Balance Shields (Ref. 142)

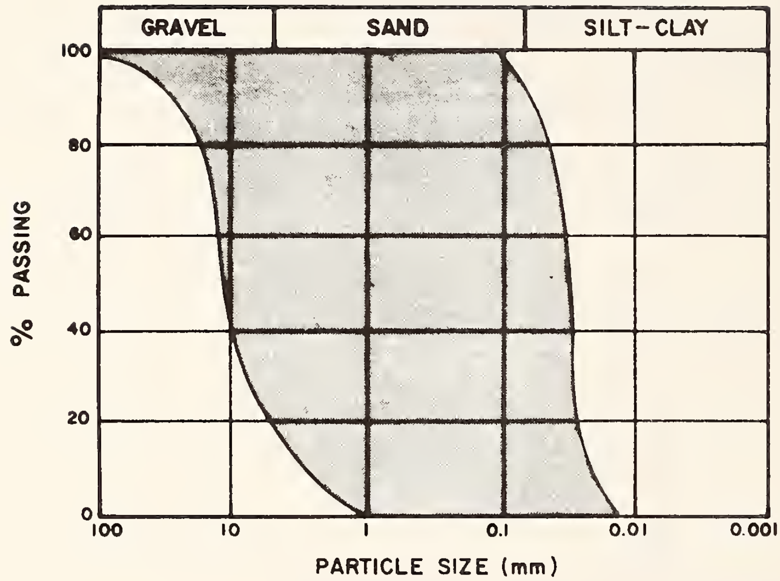


FIGURE 38 - Soil Gradation Guidelines for Efficient Operation of Slurry and Earth Pressure Balance Shields (Ref. 146)

Development of slurry shields and earth pressure balance shields has been primarily a Japanese effort, although they credit the original idea to the British. Significant work has been done by German workers as well. While it is reported that work is being done by an American company on development of an earth pressure balance shield, the Japanese are still the leaders in this technology. The first use of an earth pressure balance shield in the U.S. was in a portion of the North Shore Outfalls Consolidation Sewer in San Francisco constructed during 1980.

In 1964, the British consulting firm of Mott Hay and Anderson, under the direction of John Bartlett, developed and patented a slurry shield tunneling machine which was used on a trial drive excavated during 1972. This trial drive was well documented and a great deal was learned by the British on slurry shield tunneling methods (Ref. 50).

The Japanese, by comparison, first worked with slurry shield techniques in 1963 and began experimental work in 1965. Ground conditions in much of coastal Japan are well suited to slurry shield methods, and, therefore, several manufacturers and contractors have experimented with the technique. By mid-1979, the Japanese had built approximately 160 machines varying in size from 3.9 feet (1.2m) to 27.9 feet (8.5m) in diameter, and it is reported that a 32.8 foot (10.0m) machine is planned.

The first German slurry shield was used to construct a 3.8-mile (4.5km) long by 11.9-foot (3.6m) I.D. sewage collection tunnel in Hamburg-Wilhelmsburg. The contract, which began in 1974, was undertaken by a joint venture of German contractors including Wayss & Freytag-Dyckerhoff & Widemann-Hicktief. The German machine is known as the Hydroshield (Ref. 147).

4.81 Slurry Shields

Machines developed in Britain, Germany and Japan vary in detail, but in principle of operation they are similar. Details of operation and design are adequately described in several references. (Refs. 5, 21, 41, 50, 254)

The slurry shield machine is a full-faced, partially closed cutter head machine designed to permit excavation while simultaneously maintaining a slurry pressure on the face (Figure 36). The cutter wheel is typically driven by hydraulic or electric motors. Most are center shaft driven, but some newly developed types utilized an outer wheel driving system which simplifies muck discharge and provides a less obstructed interior working space. Fresh slurry is pumped to the face and muck is removed in a slurry return line which runs to a slurry cleaning plant

at the surface. The Japanese and British use a system of pumps and sophisticated instrumentation to maintain slurry pressure, while the Germans use a double compartment system involving a compressed air/slurry interface. The Germans claim that the air controlled system is superior to the British and Japanese flow controlled systems because it is simpler and less liable to failure. The pressure can reportedly be maintained by hand in the event of a circulation system failure. A limitation of the German machine is that it cannot be used in tunnels less than about 13 feet (4.0m) in diameter.

The Japanese and British cutter wheels are similar in that they are aligned perpendicular to the shield axis and have muck slots of limited size in the face plates. The British designed cutter wheel is slightly beveled as opposed to the typically flat-faced Japanese design. The German machine also has a slightly beveled cutter head, but it is mounted on a shaft that is inclined slightly downwards relative to the shield axis. The cutting tool, in contrast to the Japanese and British machines, is an open wheel with cutting tools designed for both directions of rotation. The Germans rely entirely on the slurry for support, while the Japanese and British have cutter wheel face plates that can provide some support in case of loss of slurry.

Both the British and Germans use a bentonite slurry, while the Japanese have used bentonite infrequently. In general, the Japanese use the fine soil being excavated to form the clay slurry with additives such as Carboxy Methyl Cellulose to prevent particles from settling out of the suspensions and also to maintain slurry viscosity. They try to avoid bentonite where possible because of its very high price in Japan, and the difficulty of disposal of bentonite-contaminated soil.

The stability of the tunnel face is believed to be dependent on the formation of a slurry cake on the soil face. This cake, if it forms, is however constantly being removed as soil is excavated. There is, therefore, some controversy as to the true nature of slurry support at the face. Some believe that support is derived by infiltration of slurry into the face.

Slurry is mixed, circulated and cleaned by means of a complex system of piping, pumps, mixers, instrumentation and sedimentation devices. Removal of excavated material from the slurry starts with simple sedimentation through coarse screens (up to 4 inches [100mm]) to progressively finer screens. Sludge is removed by means of coagulants, centrifuges and vacuum filters. The slurry cleaning process is a major consideration in the use of slurry shields because of the time involved and because of the physical size of the cleaning apparatus at the surface. Work is continuing on the development of more compact units.

As with any construction method, there are advantages and disadvantages to the use of slurry shields. These are summarized below:

Advantages

1. Use of pressurized slurry at the face allows for tunneling in water bearing soils.

2. Control of the face by pressurized slurry minimizes surface subsidence when compared to other conventional tunneling methods. The Japanese report that they are typically able to limit surface settlement to less than 4 inches (100mm).

3. There is essentially no chance that a blowout can occur as with compressed air and, therefore, it is possible to drive tunnels through saturated granular soils with relatively shallow cover. Tunnels with as little as 6 to 10 feet (2 to 3m) of cover have been driven successfully.

4. Because compressed air is not used, the tunnels are safer and present greatly reduced health hazards. Worker productivity is improved because men are working in free air. The biggest benefit is a reduction in the amount of expensive labor which is required in compressed air tunnels.

5. A more accurate alignment can be maintained than is possible with other mechanical shields because the slurry in the heading reduces soil drag on the shield.

Disadvantages

1. The machines typically have difficulty handling gravel and cobbles over 2 to 4 inches (50 to 100mm) in size. Cobbles up to 7.8 inches (200mm) have been handled by means of special techniques, but it is difficult. This can be done by passing the discharge through a trommel immediately behind the shield to remove the larger cobbles. This requires a break in the slurry fluid circuit and may cause some circulation control problems. Another method is to build a small rock crusher totally enclosed in the hydraulic system behind the shield and ahead of the slurry discharge pumps.

2. Sealing the space between the shield and tunnel lining is difficult, and slurry leakage has been a continuous problem. Numerous tail seal designs have been tried,

including wire brushes, multi-filament urethanes and a variety of robust rubber products. Some designs are illustrated in Figures 39 and 40. Continued improvement is being made in tail seal designs, and this is not longer a significant problem. The current design trends are to more flexible rubber flat strips with sealing pressure applied by numerous overlapping pieces of spring steel. The practical limit on length of drive being attempted with one set of seals increased from about 2600 feet (800m) to 6500 feet (2000m) in the period from 1976 to 1980, and the maximum pressure that can be successfully sealed increased from about 1.0 atmospheres (98 kPa) to about 2.5 atmospheres (245 kPa) in the same period.

3. Soils with a coefficient of permeability of 10^{-2}cm per second or greater can result in significant slurry loss with resultant loss in stability.

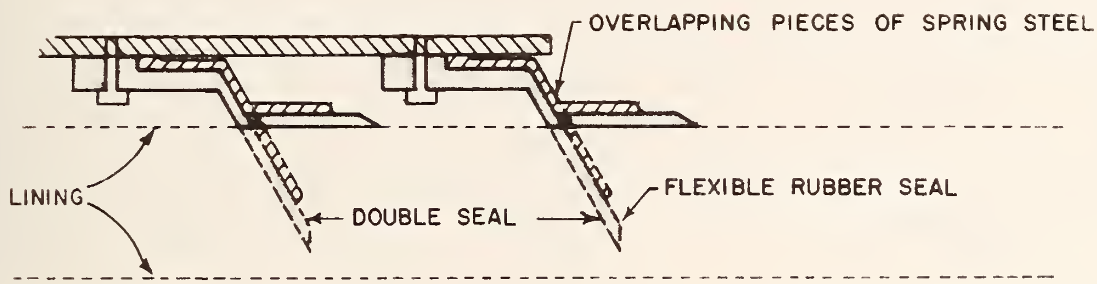
4. Slurry treatment requires a large-scale surface plant which can be a difficult problem in congested urban areas.

5. As with any highly mechanized full-face tunneling system, the system is vulnerable to changing ground conditions. Slurry shields do not operate well through cobbles, nor do they operate well in highly plastic soils, both of which tend to clog the muck handling system.

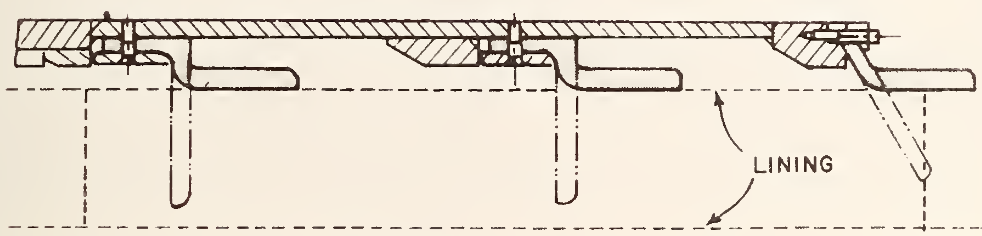
The slurry mole is a very useful tunneling machine for soft ground tunneling within a relatively narrow range of soil conditions, i.e., saturated fine gravel, sand and silt. It does not perform effectively in cohesive soils or if cobbles and boulders are encountered. At the present time, the Japanese are clearly the technology leaders, although the British and Germans also have experience.

4.82 Earth Pressure Balance Shields

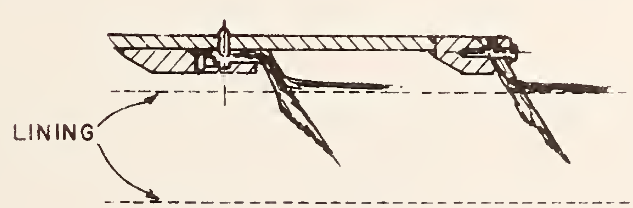
The Earth Pressure Balance Shield (E.P.B.S.) (Figure 37) is similar in principle to the slurry shield; however, face stability is maintained through controlled removal of excavated material from the face. The Earth Pressure Balance Shield is a by-product of the continuing search by Japanese tunnel designers and constructors for a soft ground tunneling machine simpler in design to the complex slurry shields. Many Japanese engineers consider the E.P.B.S. to be a better system than the slurry shield.



SPRING STEEL AND RUBBER



W-TYPE TAIL PACKING



DOUBLE WIRE BRUSH TAIL PACKING

FIGURE 39 - Japanese Tail Seals (Ref. 281)

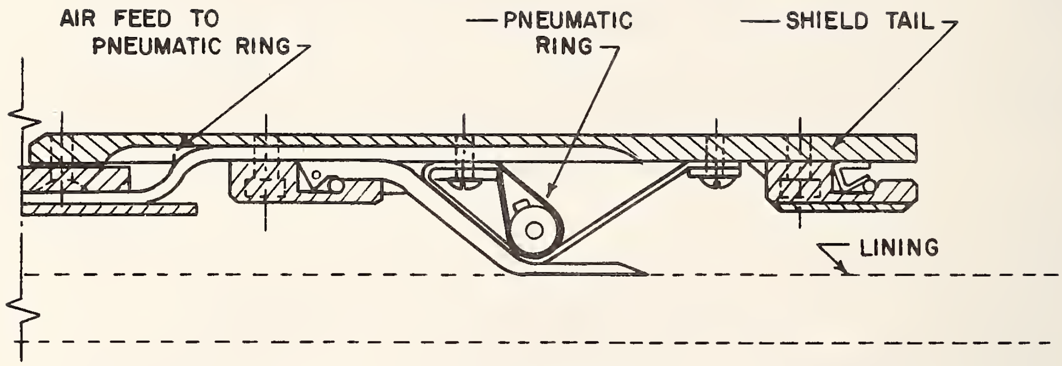


FIGURE 40 - A British Tail Seal (Ref. 50)

Excavated material is stored in a drum wheel at the cutting face. The excavated volume is balanced with the disposed volume by accurately controlling the face pressure, cutting torque and rotation of the screw conveyor. The earth pressure at the face is, therefore, balanced against the soil pressure maintained in the drum and screw conveyor housing which constitute a single sealed chamber. As with the slurry shield, critical controls are provided through sophisticated automatic monitoring and control systems. To date, only the Japanese have built earth pressure balance shields.

There are two variations of the earth pressure balance shield, which are the water pressure type and the soil pressure type, as illustrated in Figure 37.

The water pressure type is designed to permit the application of a water pressure in the drum and screw conveyor housing to counteract external hydrostatic pressure. It is not used widely and is believed to have been developed during the development of the soil pressure type. The mucking adjuster, screw conveyor, and drum form a pressure vessel. Muck is mixed with clear water in the mucking adjuster, forming a natural slurry which is pumped from the tunnel in a manner similar to that used in the slurry mole.

With the soil pressure type shield no slurry is created and, therefore, excavated soil can be removed by conventional rail haulage or conveyor systems. This method has the distinct advantage that no large slurry cleaning plant is required at ground surface. This allows for a smaller crew requirement, and the controls can be placed at the shield instead of on the surface, as required with the slurry shield.

The earth pressure balance shield, whether of the water pressure type or soil pressure type, is subject to most of the same advantages and disadvantages as the slurry shield. Problems with adequate tail seals, complex control systems, and system inflexibility to work effectively in cohesive or cobble-strewn soil plague all of these shield types, i.e., slurry and earth pressure.

It appears that in years to come, slurry or earth pressure shields will be used in lieu of compressed air with increasing frequency. It is possible to tunnel at shallower and greater depths with these shields than is now feasible with compressed air. Earth pressure balance shields are a more flexible and simpler system than slurry shields and, therefore, will probably be used more frequently than slurry shields.

4.90 ELECTRO-OSMOSIS

A very special technique which may be applicable primarily for groundwater control in cut-and-cover tunnel excavations is electro-osmosis. The procedure is based upon the behavior of pore fluid within a soil mass across which an electric potential is established. In most soils this potential creates an electrical current where the movement of positive ions through the pore fluid towards the negative pole drags water molecules through viscous action. The amount of water movement which occurs is a function of the soil type, pore water fluid and applied voltage gradient, as well as a number of less significant factors. The great advantage of the method is its applicability in fine-grained soil such as silts, silty clays and clayey silts, i.e., soils which cannot be effectively dewatered using conventional methods.

Fluid transport by electro-osmosis is initiated by the insertion of a positive pole (anode) and a negative pole (cathode) into a soil mass, resulting in a movement of positively charged particles (cations) to the cathode. Water molecules in the voids between soil particles (and even within the double layer of finer grained particles) are dragged towards the cathode where they can either accumulate or be extracted by pumping. The procedure can thus be used to: (1) consolidate soils by reducing excess pore pressure, (2) stabilize soils by reducing water content and thus increasing strength, or (3) dewater excavations by reversing or retarding seepage. Typical successful applications in the dewatering of open cuts have been reported by several sources (Refs. 8, 54, 66, 67, 201).

A number of authors have described the theory behind the electro-osmosis phenomenon in soils. Among the earliest detailed descriptions is that by L. Casagrande in which the flow resulting from electro-osmosis is related to basic soil and pore fluid properties (Ref. 66).

Successful applications of electro-osmosis have been reported under extreme ground conditions, although no direct applications to the dewatering of tunnels have been found. Casagrande initially applied the process to the stabilization of cuts and embankments by reducing water contents or reversing the direction of seepage. The basic process involves the insertion of iron or steel rods into the ground to serve as anodes (positively charged electrodes), and well points or slotted steel pipe to serve as cathodes (negatively charged electrodes). The positioning of the electrodes depends upon the geometry of the situation, but may typically involve parallel lines of anodes and cathodes which serve to

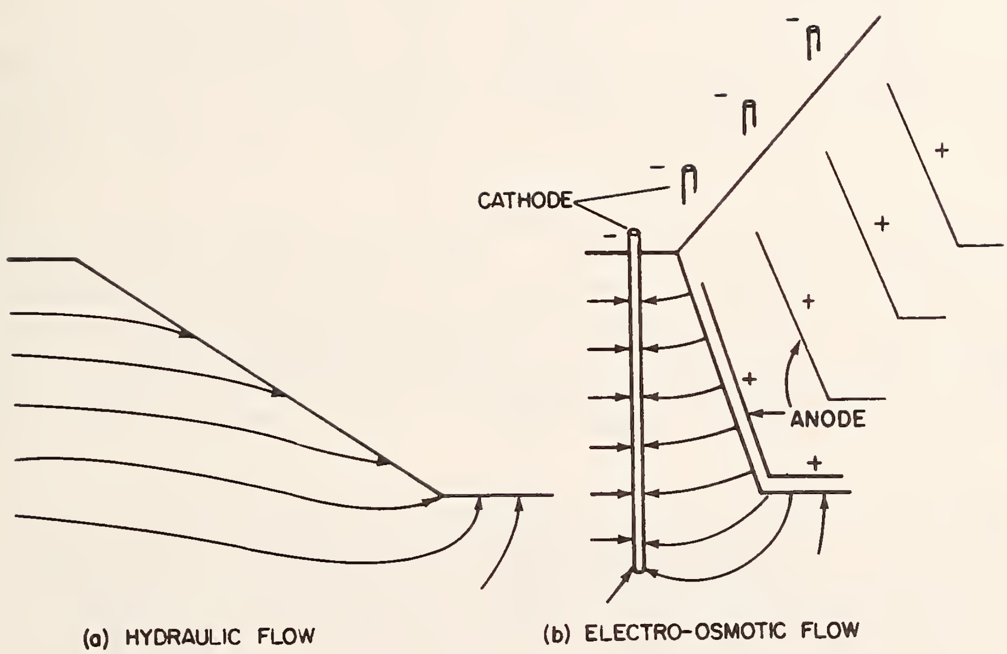


FIGURE 41 - Excavation Stabilization by Electro-Osmosis (Ref. 66)

induce seepage away from the proposed excavation (Figure 41). Where it is anticipated that large volumes of water will be pulled through the soil stratum, a pump may be used to remove it when it reaches the cathode.

The application of electro-osmosis to the stabilization or dewatering of fine-grained soils is expensive. The high cost of sustained electical power supplied to a large number of electrodes makes the process uneconomical in many situations when compared with more conventional methods. There are additional problems associated with the availability of experienced contractors and reliable direct current electrical sources.

Casagrande first showed that the rate of electro-osmotic flow is independent of the size of the soil voids and thus of great advantage in fine-grained soil. The efficiency and economics of electro-osmosis for accelerating the consolidation process, however, depend upon the volume of water transferred per unit charge, a quantity which may vary over several orders of magnitude depending on water content, soil type, and pore electrolyte concentration. For purposes of dewatering, it may be sufficient to simply reverse the direction of seepage away from the proposed tunnel, in which case the efficiency of the system in terms of water quantities may not be important. Where the soil must be strengthened to increase stability adjacent to the tunnel, consolidation and reduction in water content may become more important.

Electro-osmosis was utilized in the dewatering of an open cut in the USSR (Figure 42) (Ref. 169). The upper phreatic surface shows the maximum drawdown obtained using vacuum-assisted wellpoints after 40 days. The lower water levels were obtained after only 3 days with the use of electro-osmosis. The permeability of the subsurface material was approximately 10^{-6} cm/sec. The applications for shallow cut-and-cover tunneling projects are obvious.

Once it has been established that electro-osmosis is a possible method for dewatering a fine-grained soil, the most important consideration is probably related to cost. Since the major costs associated with this procedure are related to energy requirements, the economics depend upon the quantity of electrical power necessary to achieve the desired effect. Methods are available for determining the approximate cost of electro-osmosis, taking into account each of these factors actual operational costs are very difficult to assess (Refs. 95, 96, 112, 273). However, it is recommended that a field test be run to determine the actual behavior of the soil before any full-scale dewatering project is undertaken.

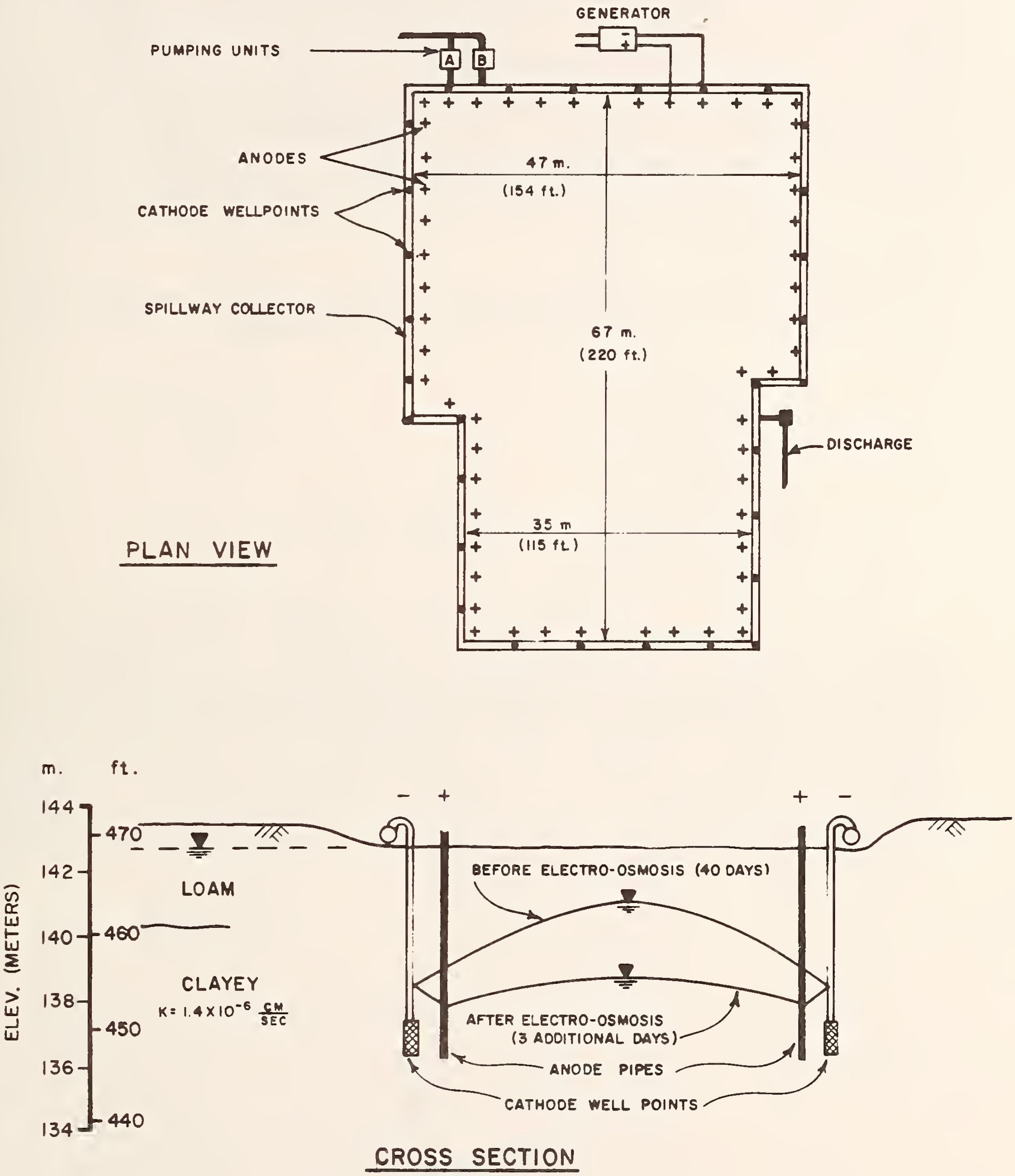


FIGURE 42 - Open Cut Stabilization in USSR (Ref. 169)

Typical spacings of electrodes have been cited as 6 to 10 feet (1.5-3.0m), although spacings as close a 2 feet (.6m) and as far as 30 feet (9.1m) have been used in instances. Energy consumption has been quoted between 0.5 and 17 KW-hr. per cubic meter of treated soil. It should be noted that these numbers are taken from sites where soil strengthening may have been as important as dewatering. For dewatering alone, the economy arises in very large excavations where the perimeter becomes smaller in relation to the volume of the excavated soil inside.

4.100 SUMMARY

Eight different groundwater control methods have been discussed in the preceding pages, including:

1. Dewatering
2. Recharge
3. Cutoffs
4. Grouting
5. Freezing
6. Compressed Air
7. Slurry and Earth Pressure Balance Shields
8. Electro-Osmosis

Each method is capable of controlling groundwater under conditions suitable to its use. Selection of groundwater control methods is based on many different technical and nontechnical criteria. One of the most significant technical criteria is soil gradation. Figure 43 is a summary of all methods discussed versus average soil types and grain sizes. The information presented is a generalization of a complex subject. It should, therefore, be used only as a guide in the process of selecting an appropriate method. A more detailed discussion of selection criteria follows in Section 5.00.

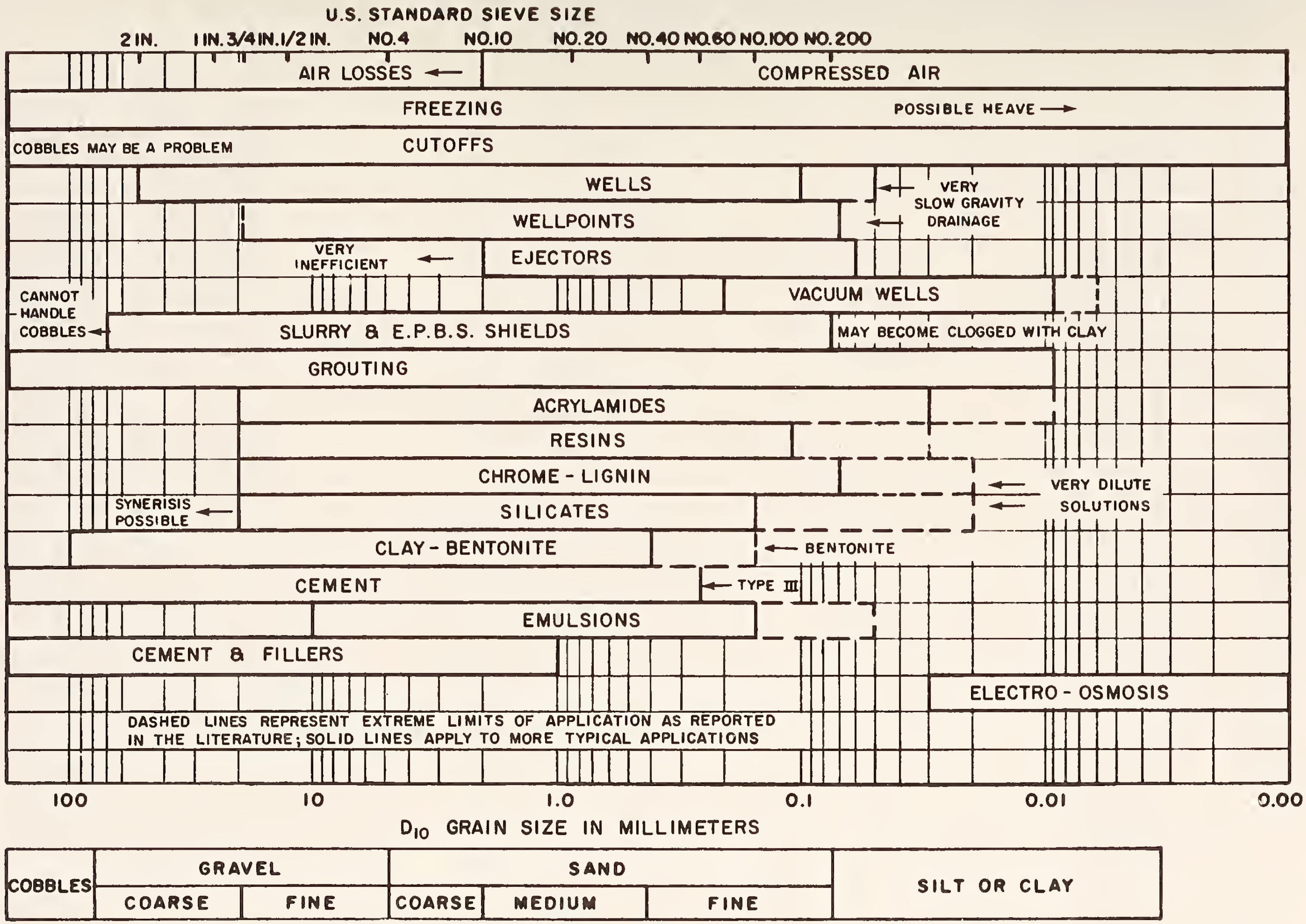


FIGURE 43 - Groundwater Control Methods vs. Soil Gradation
Compiled from a number of sources including Ref. 4, 44, 50, 66 87, 91 103, 104, 131, 133, 136, 147, 151, 165, 173, 177, 188, 234, 246, and 284

5.00 SELECTION CRITERIA

5.10 GENERAL

Selection of an appropriate groundwater control method is based on consideration of various technical and nontechnical criteria, many of which have been described in Section 4.00. Major technical criteria include:

1. Soil and rock conditions
2. Groundwater regime
3. Groundwater chemistry
4. Duration of groundwater control
5. Climactic conditions

Major nontechnical criterial include:

1. Labor practice
2. Installation costs
3. Project duraton (operational cost)
4. Effects on adjacent property and people

After proper consideration of the various technical and non-technical criteria, one must then assess risk factors. As previously mentioned in this report, groundwater control methods can be viewed as insurance policies; the greater the protection, the greater the cost. An owner or contractor may not be willing to pay the extra costs required for complete protection, i.e., closer well spacings, more grout holes, etc., and therefore the final selection is a judgement, usually by a contractor, made after consideration of the rational criteria to be described in the following sections.

5.20 TECHNICAL CRITERIA

5.21 Soil and Rock Conditions

Consideration of geologic conditions is probably the most significant technical criteria affecting selection of an appropriate groundwater control method. Permeability of individual soil and rock units, soil stratification and the possibility of encountering both soil and rock in a tunnel face must all be considered.

The effect of permeability on selection of appropriate groundwater control methods has been discussed in the descriptive material of Section 4.00 and is summarized in Figure 43. See Volume 3 for presentation of method selection matrices.

Soil stratification and the location of soil/rock interfaces relative to the tunnel strongly influence the selection process. In this section a series of possible stratification sequences and soil/rock interface positions are summarized together with comments on likely applicable groundwater control methods. The stratification is limited to possible combinations of single permeable and impermeable strata. While it is realized that much more complex stratification is possible, the sequences described illustrate basic concepts of groundwater control in stratified and mixed face conditions. Sequences to be discussed are summarized on Figure 44.

5.21.1 Uniform Soil Face Tunnel

All soils are a mixture of grain size varying from clay to gravel. Typically, a single grain size range predominates and therefore controls soil behavior, particularly in the presence of water. Selection of a technically acceptable groundwater control method under uniform conditions is relatively simple and comments for each of the major soil types, i.e., clay, silt, sand, and gravel follows.

Clay - Groundwater control is usually not a major consideration for tunnels driven or excavated in clay. The impermeable nature of clay precludes any method of grouting or predrainage, however, it can be effectively dealt with by other means such as freezing, compressed air, slurry and E.P.B. shields or electro-osmosis, which are used primarily to control stability. The possibility of heave during freezing and subsequent movement during thawing must be considered. Care must be taken to prevent excessive settlements due to consolidation of soft clays resulting from reduced pore pressure due to drainage.

Silt - Silt is a fine grained generally non-cohesive soil of very low permeability, but its stability depends on its consistency and seepage forces. Very small amounts of water under low gradients can cause the soil to run. It can be stabilized for excavation by methods that control seepage forces i.e., predrainage or compressed air, or improve consistency, (i.e., grouting or freezing).

To predrain silt is difficult, but possible, because of its permeability. The creation of a vacuum in the soil by wellpoints or ejectors can have dramatic effects in stabilizing silt. The drainage, however, usually takes a long time and therefore must be initiated well in advance of tunnel excavation. Freezing can be used but heaving may be a problem. For coarser silts, chemical grouting is sometimes feasible (Figure 43). Compressed air is also practical due to the low permeability of the soil.

If the silt is organic in nature, possible settlements must be considered. Slurry and earth pressure balance shields have been used successfully in silts, in lieu of compressed air.

Sand and Gravel - Granular soils, i.e., sands and gravels, are usually free draining with the speed of drainage dependent on gradation plus hydraulic gradient. The coarser the sand or the more uniform its gradation, the greater its permeability. Predrainage is favorable as a groundwater control method, with the type of system dependent on the permeability and hydrologic conditions. Freezing is possible as long as natural water flow is low and heaving is unlikely. Cement and chemical grouts are also an effective means of seepage control in these soils. Compressed air is applicable in finer-grained sand, but excessive air losses may result when coarser material is encountered. Slurry and E.P.B. shields are well suited to operation in saturated sands, but excessive slurry loss can occur in coarser grained materials (Figure 38). In a well-graded sand, minor seepage into the tunnel usually won't carry material since a natural filter develops at the face. This water problem can thus be usually dealt with by pumping inside the tunnel.

Gravel - Gravel is a very permeable material transmitting large quantities of water. However, the gravel is usually stable at the face even when water is flowing and therefore predraining can be rapid and effective. Compressed air is typically not feasible because of extremely high air losses and freezing may be possible provided that seepage gradients are not too high. High seepage velocities result in added heat to the system and make freezing very difficult. Similar concepts are also applicable when freezing rock formations. Thick suspension grouts can effectively control groundwater in gravels, however, addition of sand to the mixture may be required in the coarser gravels. Cut off walls can be used effectively for control of groundwater under nearly all geologic conditions except rock and very dense gravelly materials containing cobbles and boulders. Under such conditions, driving of steel sheeting or excavation of slurry trenches can be extremely difficult.

5.21.2 Mixed Face Tunnels

When more than one soil or a combination of soil and rock types is encountered in a tunnel heading, it is referred to as mixed face tunneling. Control of groundwater is essential to successful soft-ground tunneling in mixed face conditons. When only one soil is present in the face, selection of a groundwater control method is usually based on a choice between alternative methods which depends as much on relative costs as it does on technical considerations. However, the existence of a different soil layer within 10 feet (3.0m) below the invert may affect the selection of the groundwater control method.

The degree to which the mixed face is a problem in tunneling depends on the locations of permeable and impermeable layers (or soft and hard layers, etc.). Figure 44 summarizes the mixed face conditions considered below.

Type IA (Interface > 10 feet (3.0m) Below Invert)

Permeable over Impermeable

When sand or gravel exists over an impermeable soil or rock layer with the interface more than 10 feet below invert, selection of an appropriate groundwater control system can be made based on the same considerations described for uniform soil conditions.

Type IB (Interface > 10 feet (3.0m) Below Invert)

Impermeable over Permeable

The presence of an impermeable layer over a permeable layer can result in problems due to excessive hydrostatic pressure which may exist below the tunnel and therefore have to be reduced (pressure relief) to minimize the possibility of boils or heave in the tunnel invert. Groundwater control methods described for uniform soils can be used for normal excavation, however, if the upper soil is a clay or plastic silt little additional control will be necessary.

Type IIA (Interface < 10 feet (3.0m) Below Invert)

Permeable over Impermeable

If the interface of permeable over impermeable soil is located closer than 10 feet (3.0m) to the invert of the tunnel the effectiveness of dewatering methods decreases. Predrainage to the top of an impermeable layer is almost impossible, unless the interface is sloping and can drain to a low point, because of the crown which always forms between adjacent predrainage devices. The height of the crown is dependent on the permeability of the soil and the spacing between the drainage points. Therefore, predrainage devices must be spaced closely to reduce the crown height to acceptable levels. If water flows along the top of the impermeable layer, it may have to be controlled from within the tunnel. The usual predrainage scheme is to remove as much water as possible at widely spaced locations (wells) and then locate ejectors or well points as close as possible to the impermeable layer to remove the remainder of the water. Cutoff methods which penetrate into the impermeable layer, such as sheet piling

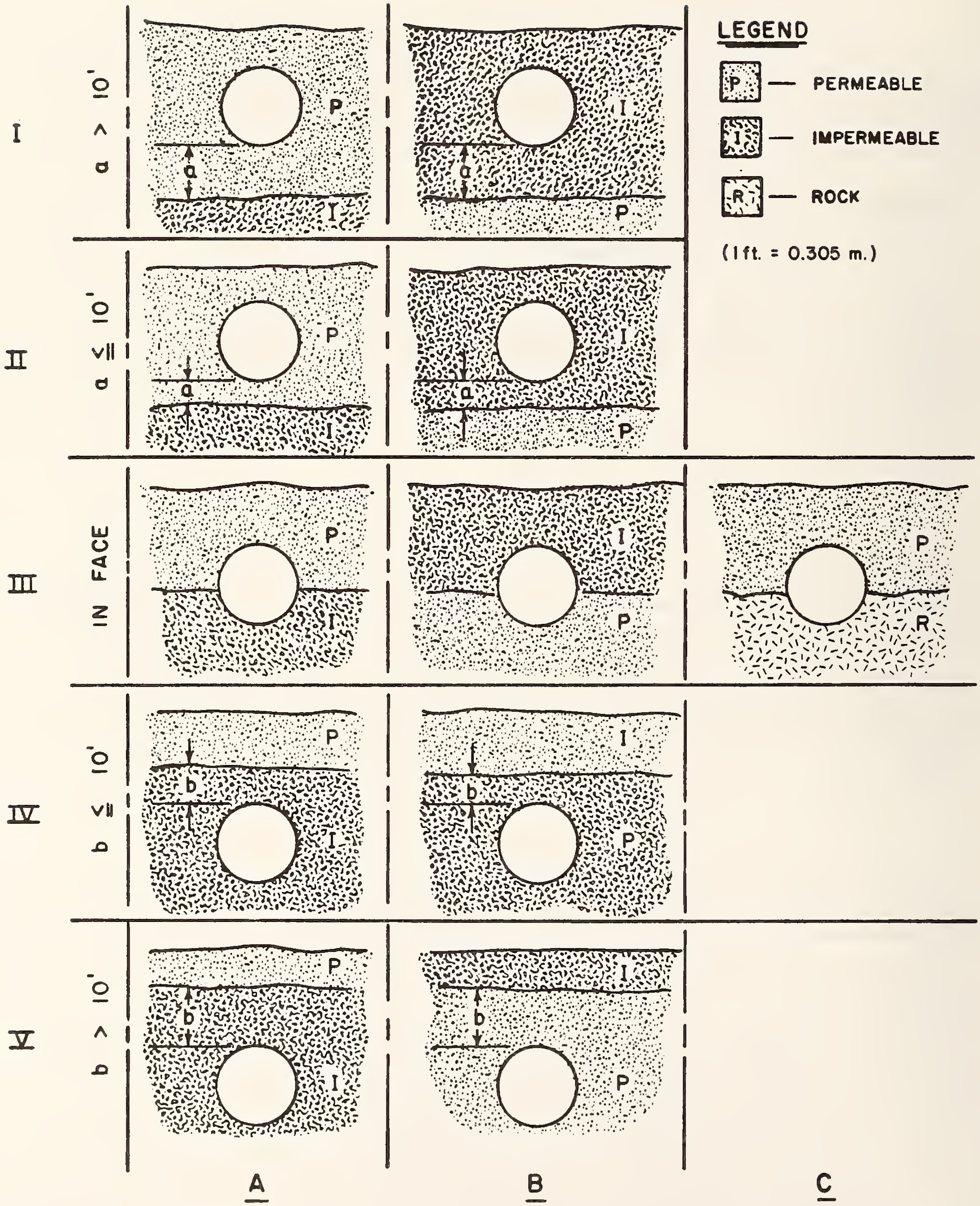


FIGURE 44 - Mixed Face Conditions

or freezing may be competitive in this situation. Compressed air may also be used, providing excessive air loss is not a problem. Other exclusion methods such as grouting or slurry shield tunneling are also technically possible.

<u>Type IIB</u> (Interface < 10 feet (3.0m) Below Invert)

Impermeable over Permeable

Control of groundwater under these conditions is similar to Type IB. The tunnel is excavated through an impermeable material and groundwater control in the tunnel is relatively straightforward. Relief of excess hydrostatic head may be necessary to prevent the tunnel invert from boiling or heaving as described above.

<u>Type IIIA</u> (Interface Within the Heading)

Permeable over Impermeable

The existence of the interface within the tunnel heading can be troublesome because excavation of non-uniform soils may preclude the use of a shield. Predraining can probably not be accomplished completely down to the top of the impermeable layer and therefore additional pumping will probably be required within the tunnel if control by predrainage is selected. The advantage of predrainage is to reduce the head of water on the material in the face to minimize face stability problems as much as possible. In this case, a loss of ground may result if the residual seepage is not controlled. Grouting has been used in this situation. Freezing is also a possibility, but pumping for predrainage may have to be stopped to avoid the continuous supply of heat. Compressed air may be feasible if excessive air loss isn't a problem.

<u>Type IIIB</u> (Interface Within the Heading)

Impermeable over Permeable

When the impermeable layer is in the upper portion of the face, the face can usually be stabilized by controlling the water below as necessary to prevent unstable conditions. Air loss is usually not a problem since the arch is impermeable. Freezing would be difficult and possibly ineffective if a cutoff cannot be obtained. However, a horizontal freeze could be considered.

Type IIIC (Interface Within the Heading)

Permeable over Rock

This situation is similar to, but potentially more difficult than Type IIIA because the tunneling method includes rock excavation. The stability of the material in the arch with respect to blasting or other methods of rock removal must be considered. This represents a difficult tunneling situation in which grouting can be extremely effective. Depending on the relative size of rock fissures and soil permeability, a two stage grouting process of suspension grout (cement or bentonite) followed by chemical grout (silicates) can be effective. Freezing has also been considered in such cases, though freezing of rock may be difficult.

Type IVA (Interface $<$ 10 feet (3.0m) Above Arch)

Permeable over Impermeable

When the interface exists just above the arch of the tunnel, groundwater control may be achieved with relative ease. The water in the overlying pervious layer should be controlled in case this stratum is intercepted which could result in excessive loss of material and possible "daylighting". The tunnel itself is in uniform material and, therefore, groundwater control is relatively simple.

Type IVB (Interface $<$ 10 feet (3.0m) Above Arch)

Impermeable over Permeable

When the complete face is in pervious material with impervious over the arch, a choice of methods is usually present. Compressed air loss is minimized by the confining layer. Predrainage is usually successful and less expensive. The effect of dewatering below a clay or silt layer may result in compression though, and analyses as to the magnitude of possible settlements must be made. Recharge may be necessary to control settlement.

Type VA (Interface $>$ 10 feet (3.0m) Above Arch

Permeable over Impermeable

This is equivalent to a uniform impermeable material and can be handled by methods stated earlier.

Type VB (Interface > 10 feet (3.0m) Above Arch)

Impermeable over Permeable

This is equivalent to a uniform permeable material and can be handled by methods stated earlier.

Since the stratification in a mixed-face tunnel is so important to the type of groundwater control selected, it is important to know what condition exists prior to construction. Once a commitment is made to a singular groundwater control method, it is usually very expensive to change when the tunnel is under construction. Flexibility is needed in mixed-face tunneling because the limitations of a singular groundwater control method may be too great if the interface moves vertically only a slight amount.

5.21.3 Rock Face Tunnel

Groundwater problems in rock tunneling differ from those in soft ground in several respects. Usually, stability of the face is not a problem. Any water entering the tunnel is, for the most part, clean and does not present a potential for loss of ground. The quantity of water that can safely enter a rock tunnel is dependent on the capacity available to pump it out. Permeability of rock masses is dependent on intact rock permeability and rock discontinuities such as faults, joints, or solutioned zones. Intact permeability depends on the material from which the rock is made and its cementation. For most rock types the intact rock permeability is usually insignificant. Rocks which can have significant intact permeability are porous sedimentary rocks such as certain coarse-grained sandstones. Flow through intact rock usually does not represent a problem in tunneling unless the cementation is weak and can possibly be destroyed by the movement of water.

Fracture permeability normally controls groundwater flow in rock masses and can vary from nearly impermeable basalt formations to extremely permeable solutioned limestones and dolomites. Faults usually are the most permeable fracture zones. Rock in the vicinity of a fault may be completely reduced to a gouge by grinding and fracturing. Flows from such zones can be catastrophic. Solutioned rock can also lead to catastrophic inflows of water. Limestones and evaporites are the most common types of solutionized rock.

Differences in rock permeability are measured in orders of magnitude much the same as soil, however, variations can be extreme over short distances. The success of groundwater control depends upon the number of discontinuities which are intercepted or sealed. Conceivably, a well or grout curtain

could be drilled 10 feet (3.0m) from an extensive solution zone and have little effect on changing groundwater conditions. The costs involved with handling the water outside of the tunnel by any means is usually much greater for rock than for soil conditions due primarily to the greater costs associated with rock drilling.

5.22 Groundwater Regime

Groundwater control methods, with the exception of recharge, can be categorized as extraction methods or exclusion methods. A thorough knowledge of the groundwater regime is necessary for the selection of a method in both categories, but it is particularly essential for the selection of an extraction method. Some of the most important aspects in the design of groundwater control systems are: nature of the aquifer, areal extent of aquifer, barrier boundaries, sources of recharge, and existing water levels. The significance of each of these aspects is described in the following sections in order to assist the reader to develop insights into why they are important.

Nature of the Aquifer

Ideal aquifers are classified as confined (artesian) or unconfined (water table). Real aquifers exist in a broad range from confined to unconfined. Aquifers within these limits are generally called semi-confined (leaky) or semi-unconfined, with no specific limits defining each category.

Confined aquifers have piezometric levels which are above the top of the aquifer. Depending on the pressure in the aquifer, unsafe conditions could exist in the tunnel excavation. To excavate safely, these pressures must either be balanced or relieved in some way. Boils and/or heave are problems associated with excess confined pressure. Frequently the pressure is relieved to safe levels by pumping. Response characteristics of a confined aquifer under pumping stress include almost an immediate reduction in pressure, and response over large distances, i.e., thousands of feet. Immediate response to changes in pumping, however, can present serious problems. For instance, failure of a pumping system would result in rapid increase in hydrostatic pressure to normal levels which could represent significant problems. Care must be taken to prevent system failure by providing an emergency power source plus standby pumping capacity. If the tunnel happens to intercept the confined aquifer, significant additional pumping may required to drain the aquifer to safe levels. Instead of reducing only pressure while keeping the aquifer saturated, drainage of a confined aquifer is similar to dewatering an unconfined aquifer.

Unconfined aquifers have water levels which are within the aquifer, i.e., the aquifer is not saturated over its full depth. Lowering of the water level in this type of aquifer involves depleting storage which is time consuming and, hence, response to pumping does not occur rapidly. Therefore, pumping must begin well in advance of excavation to allow this storage depletion to take place. A temporary failure of the pumping system does not cause immediate rise in water levels since a portion of the water goes back into storage. This slower response to pumping allows overpumping for periods of time so that occasionally the system can be started and operated on a shift basis rather than continuously. In addition, the drawdown due to pumping is usually not evident over as great a distance as for a confined aquifer.

The combination aquifers, those somewhere between confined and unconfined, exhibit properties of both, however, not to as large an extent. It is difficult to make general statements about these aquifers. Usually the best way to predict response is through extrapolating results of field pumping tests.

Permeability of the Aquifer

Permeability is an important criteria in the selection of any groundwater control method, but it is especially critical to drainage schemes. When the permeability is low, water cannot pass readily and so the effect of drainage is to produce a steep gradient toward the drainage location. The radius of influence of a well in such an aquifer is relatively small. The amount of water that can be pumped is low, though it is directly dependent on the length of submerged well screen during pumping. To obtain required drawdown between two drainage devices requires closer spacing. The steep gradient produces a higher crown of the water level between devices.

When permeability is high, water flows more easily to the drainage location. A flatter gradient develops radially from the well. Therefore, the radius of influence is greater and the amount of water to be pumped to obtain a given drawdown is greater. However, because the crown is flatter, adjacent devices can be more widely spaced.

The basic considerations of aquifer permeability are then useful in determining: number and spacing of required drainage locations, amount of water that is to be pumped at each location, and the radius of influence of the installed pumping system.

Depth of Aquifer

The depth of the aquifer below tunnel invert is often not determined prior to bid. Borings are usually widely spaced along the alignment of the tunnel and extend a predetermined depth below invert. This is often not sufficient for the design of a groundwater control system. For example, in the design of a predrainage system it is necessary to know how deep the aquifer is in order to estimate both well capacity and optimum spacing. Well capacity is directly dependent on the amount of submerged screen remaining in a well in its operating condition. A deep aquifer usually necessitates pumping a larger volume of water for a given required drawdown than if the aquifer is shallow. Other considerations requiring a knowledge of the aquifer depth include determining the cut-off tip elevations for sheeting, freezing or slurry wall design.

Areal Extent of Aquifer

Boundary conditions are important to the hydrologic design of predrainage systems. The "ideal" aquifer is infinite in areal extent and the theoretical radius of influence, assuming no recharge, expands to infinity with time. However, if a zone of recharge or a barrier to flow exists, the pumping system response will depart from the ideal situation.

Recharge is important in both confined and unconfined aquifers. Since confined aquifers are separated from the atmosphere by impermeable layers, recharge from local sources does not exist. Usually the recharge source for a confined aquifer is located at a great distance from the pumping system, which is one reason why the normal radius of influence in a confined aquifer is so great. However, recharge sources for unconfined aquifers are usually not very distant. Local precipitation replenishes the aquifer as do surface water sources which may be hydrologically connected to the aquifer. In principle, the radius of influence expands until the amount of water entering the cylinder bounded by the radius (the recharge flow) is equal to the quantity of water being removed by pumping. However, any sources which are within this natural radius will distort the regime and require that a larger quantity be pumped to achieve a given drawdown condition.

Barrier boundaries result in the appearance that the effective radius of influence is further away than expected. The apparent greater than actual radius of influence can result in over-design of a dewatering system.

Therefore, to accurately design a pumping system, knowledge of both barrier and recharge boundaries is necessary.

Existing Water Levels

Most investigations produce basic data on existing groundwater levels. These data are important because they help define the magnitude of the groundwater problem. The amount of pumping required for a predrainage system or extent of a cut-off for an exclusion system are indicated by existing water levels. They are also important for the design of support systems and prediction of settlements. Approximations of aquifer parameters can sometimes be made without the aid of pumping tests by careful examination of water level fluctuations. Natural gradients across a site can indicate recharge and discharge locations.

Groundwater Chemistry

Accurate water chemistry analysis is essential to design of a groundwater control system. Lack of such information frequently leads to bid price contingencies for special materials, or frequent system maintenance; or if not, becomes a basis for claims for changed conditions.

Adverse water quality usually manifests itself as an encrustation or a corrosion problem in predrainage systems. The major effect is a reduction in system performance by either decreasing intake capacities, or by clogging of the screens, pumps or piping. In order to avoid these problems, special material must be used in the construction of the drainage device. In both cases the usual material chosen is a plastic which is not subject to corrosion and also is resistant to the acids which may be used to dissolve encrustation deposits. Special pump components may be required to protect against corrosion, usually at a substantial cost premium.

Regular maintenance must be performed on predrainage systems to remove encrustations and to deep the system performing properly. In severe cases, costly continuous maintenance may be required.

Recharge wells are very sensitive to water chemistry. Encrustation is usually more severe than in the predrainage systems. Maintenance can be costly and difficult since the precipitate is being carried into the filter and surrounding aquifer, whereas in predrainage systems the deposition is usually confined to the screen.

Ejector systems are also very sensitive to water chemistry. A slight encrustation or corrosion of the nozzle and venturi can significantly decrease the efficiency of the system. Ejector systems selected based on hydrologic considerations can be totally unworkable in practice due to groundwater chemistry problems.

Disposal of poor quality water creates environmental concerns which can significantly increase costs. If water is to be discharged into existing streams or ponds it should be chemically compatible with the receiving water body and may thus require pre-discharge treatment.

In slurry walls, slurry shields and grouting schemes, groundwater quality can be detrimental to grout and slurry properties. An investigation of grouting slurry stability in the presence of the natural groundwater should be undertaken if these methods are considered.

5.23 Duration of Groundwater Control

Duration of groundwater control must be considered to insure the most economic material to use for construction and to estimate effects on people and property as discussed in Section 2.40. For example, dissolved iron in the groundwater can be controlled in the short-run but encrustation building up can cause gradual loss of system capacity in the long-run. If long-term groundwater control is required, the cost of a maintenance program to restore pumping efficiency may be considerable. In corrosive environments, pumping components may be able to withstand the corrosion for a short time without loss in efficiency. However, a long-term requirement may necessitate special, corrosion-resistant components at what is usually a large premium.

Under a long-term groundwater control requirement, even normal wear and tear must be considered in keeping spare equipment on hand. Most cut-off methods are insensitive to duration of groundwater control, with the exception of freezing which incurs time dependent operating costs.

5.24 Climatic Conditions

The effect of precipitation and temperature on groundwater control methods must be considered because climatic conditions can add considerable cost to the system.

Seasonal fluctuations in temperature affect groundwater control systems in different ways in different parts of the country. For example, in Buffalo, groundwater control is being accomplished through the use of predrainage wells. Due to the severity of winters in Buffalo, the wells are placed in vaults beneath the streets. These vaults are equipped with electric heaters to prevent freezing of the groundwater at the wellhead. The protection of any dewatering system from freezing must be designed prior to installation of the system. In cases where there is a likelihood of standing water in pipelines, either due to a pump failure or lack of sufficient flow, the piping system must be designed to drain by gravity flow, or be insulated.

Heavy precipitation can effect the design of dewatering systems. The infiltration of rainwater into an aquifer may cause static water levels to rise significantly. However, most of these fluctuations tend to occur on a seasonal basis. If such fluctuations are anticipated, the dewatering system must be designed to account for them. Fluctuating water levels may be handled by installing extra capacity in the dewatering system to be used on a standby basis when required.

In confined aquifers, dewatering systems are usually specified to protect against pressure increase due to some specified maximum pressure. Protection against floods of short duration can be achieved by depleting storage in advance of a rising river stage. When the river rises, part of the water will go to refill storage thereby reducing any rise in groundwater level. With proper scheduling, a significant savings may be realized because of reduced need for additional pumping capacity.

5.30 COST CRITERIA

Once all the technical criteria have been considered, the cost of possible methods of groundwater control must be compared. One of the main objectives of this report is to indicate how adequate groundwater control may be obtained at the least cost. However, items which affect the cost of a groundwater control system are so varied, site-specific and inter-related with other construction considerations that the cost of groundwater control per foot of completed tunnel has been seen to vary by several orders of magnitude from project to project. Presentation of typical unit costs or rules of thumb can be extremely misleading.

Depending on the type of groundwater control system considered, items affecting the cost of that system usually fall into one or more of the following areas: mobilization, installation, operation, technical and other considerations. Often, after weighing all these elements of cost, the best groundwater control system technically will be set aside for a less desirable one but is more economical to install, operate or maintain.

5.31 Mobilization

A. Availability of Equipment - Specialty equipment may be difficult to acquire locally, requiring it to be shipped great distances. Perhaps a piece of the contractor's equipment may not be available when it is needed and he will have to rent the equipment from outside sources at premium cost.

B. Availability of Material - A special filter required by a well design may not be available locally, requiring special manufacturing or increased shipping charges. Special corrosion resistant materials may not be readily available and/or cost a great deal to fabricate.

C. Access to Site - Can the groundwater control system be easily installed or is there an existing building at the location of a critical well? Examples where access may be a problem are: courtyards, pedestrian shopping areas, private property, parks, and historical sites.

D. Density of Utilities - Overhead clearance for certain equipment may require the relocation of utility lines. Buried utilities may prevent the installation of certain groundwater control devices in some cases and may make their installation more difficult in others.

5.32 Installation

A. Drilling Problems - Availability of water for drilling may affect cost. In some locations, the use of fire hydrants or water mains is permitted while in others water must be hauled in tank trucks to the drilling site. Difficulty in penetration has a major impact on the progress of the work. Disposal of cuttings and drilling upon completion of the hole must be considered.

B. Labor Work Rules - Union rules vary from local to local and it is imperative that the work rule ageements be known before estimating the cost of the work. For example, currently in New York City a crew of five maintenance engineers and one apprentice engineer is required for the installation of wells. In some other locations a well may be installed using only the driller and his helper.

C. Work Areas - If there is an area on the site where equipment and material may be stored and stockpiled, the work is usually performed more smoothly and with less interruption. If the only available storage area is at some distance from the work site, delays become an important part of the estimate. The delays may be due to distance, traffic, and general inconvenience.

D. Local Restrictions - Consideration must be given to any special requirements, such as maximum permissible noise levels, limitations on work times, on traffic maintenance, or on work locations. For example, in Cambridge, Massachusetts, if work must be performed on a main street, work on the in-bound traffic lanes may not start until after morning rush hour, and on the out-bound lanes it must be completed prior to afternoon rush hours.

E. Special Requirements - Sometimes requirements placed on the surface construction of a groundwater control system are such that these become a major, if not the main, cost element of the entire system. Example of these requirements may be burying discharge and header pipes, installing the wells in manholes or vaults, lack of nearby discharge points, and special protection to prevent vandalism. On the NFTA project in Buffalo, New York, the expected quantity and quality of the discharge water was such that several thousand feet of buried discharge pipe was installed as well as a chemical treatment plant to handle the discharge flow. Depending on the type of groundwater control system and the location of the site, protection against extreme heat or cold may be required.

F. Depth - The cost of almost every groundwater control method increases with increasing depth of installation. This results from the increased time required for the installation, additional material costs, and possible, larger installation equipment requirements.

5.33 Operation

A. Labor Work Rules - It is imperative to know local union agreements when estimating the cost of operation. For example, in New York City the current agreement is that a maximum of five submersible pumps may be operated by one operating engineer per shift. This can have a profound effect on the actual number of wells to be installed. A primary objective of a well system design is to select the minimum number of wells to be operated simultaneously to produce the required drawdown at any tunnel location. In cases where a total of more than five wells are required for the entire tunnel, the restriction to only five wells operating at any one time will require close spacing of wells in order to achieve the same cumulative drawdown within the smaller tunnel segment affected by these five wells. Thus, many more wells than technically necessary may have to be installed over the length of the project because drawdown requirements must be met by operating no more than five, rather than all wells. Under these conditions good design must incude balancing this incremental operating labor cost against the cost of installing additional wells at closer spacing. In designing for cut-and-cover tunnels, additional flexibility is often possible by adjusting excavation schedules so as to accommodate the five-well restriction.

B. Duration of Groundwater Control - Besides the obvious costs of fuel or electricity to run a groundwater control system, the duration of required control is important in other respects. For example, exclusion (cutoff) methods of groundwater control often have higher installation costs than extraction (pumping) methods. But extraction methods have an operation cost which

continues through the life of the project, while the operation cost of a cutoff method is negligible, except in the case of freezing. The longer the duration of the project, the more competitive the two metods become, without consideration of secondary functions of the cutoff, such as structural.

C. Water Chemistry - If the groundwater is corrosive or scale-forming, additional costs may be incurred. Cleaning and maintenance cost of the system may be greatly increased over the usual allowances. Special noncorrosive materials may be necessary in the control system components. Treatment of the pumped water may be required before it can be discharged into existing sewers or water bodies. Recharge systems are especially susceptible to problems of water chemistry. Dissolved iron in the groundwater may make the utilization of an ejector pumping system very expensive or even impractical.

D. Discharge Restrictions - Groundwater which is pumped may not be of acceptable quality for discharge into surface bodies of water, requiring special treatment prior to discharge. Occasionally existing sewer systems cannot handle the quantity of water expected to be pumped. Municipal treatment facilities may charge for the conditioning of this discharge water. Occasionally, limits are placed on the quantity of water which may be pumped under diversion permits. If the required pumping is more than is permissible, recharge may be required to replace pumped water.

E. Safety Factors - Safeguards should be built into the groundwater control system to protect against possible interruptions in power supply or failure of portions of the system. These safeguards may be in the form of standby diesel generators which may be utilized upon any interruption in the normal supply of electricity. There may be redundancy in system design to allow for temporary failure of one or more pumping units without jeopardizing the ability to maintain the required drawdown.

F. Power Costs - Especially in projects of long duration, minimizing required horepower and maximizing the efficiency of the pumping system is of great importance. Proper design of the electrical distribution system can have major effect on operating costs.

5.34 Technical Factors Affecting Cost

A. Quantity of Water - Large quantities of water may make an ejector system impractical, necessitating a shift in design toward a well system. If the quantity of water is minimal or its occurrence is sporadic, it may be cheaper to treat the water problems as they occur rather than pay for design and

installation of the groundwater control system in advance of tunneling. This decision must reflect a balance of the cost of advance installation versus the probability that the system will effectively intercept all potential seepage flow.

B. Required Drawdown - The cost of all dewatering schemes increases with increased drawdown requirements because of the need for increased horesepower, multiple wellpoint stages, etc. Partial lowering of the groundwater level is common to allow compressed air tunneling at pressures less than one atmosphere so that productivity can be increased, which results in more economical construction.

C. Stratification - Soil which is stratified with frequent impermeable layers does not permit groundwater to drain readily in the vertical directions. Therefore, pumping methods in these soils may yield mixed results. Water perched on top of these impermeable layers may or may not present problems to the contractor depending upon whether or not he is prepared to handle this seepage into the excavation. It may be necessary to resort to grouting, freezing or some cutoff method to achieve effective groundwater control with attendent cost implications.

D. Nearby Source of Recharge - Where the source of recharge to the aquifer is nearby, the design of a groundwater control system requires additional safety factors. For example, the groundwater control systems near a river are often designed to maintain drawdown at maximum river stage. If the source of recharge to the aquifer is extremely close, full or partial cutoff methods may be required to augment the pumping system at increased cost.

6.00 LEGAL AND CONTRACTUAL CONSIDERATIONS

6.10 GENERAL

Much of this report has catalogued the many difficulties in tunneling which can be related to groundwater control. Because these difficulties are often not anticipated, or if anticipated are not subject to accurate prediction, they generally result in significant increases in cost. Much of this cost increase is incurred during the bidding process, when the contractor must make his best assessment of the risks associated with groundwater control and incorporate the corresponding contingency costs in his bid.

In the case of bidding for a major urban tunneling project, it is likely that 90% of the bidding contractors have total per diem costs (construction plant, labor and equipment) that are essentially the same. These costs include those of the groundwater control method. The real competition in the bidding resides in their individual estimates of expected progress (feet of tunnel driven) per day. This estimate is a comprehensive assessment of all risk and uncertainties perceived by the bidder. It may reflect compound uncertainties, such as the probability of encountering both boulders and groundwater problems, where neither obstacle is reasonably predictable from data in the bidding documents.

Acknowledging that risk is always embodied in groundwater control, it becomes the function of the legal and contractual arrangements for the tunneling project to determine how the several parties to the tunneling project shall share the burden of these risks. In the following sections we shall examine which of the parties are most closely related to various elements of risk and how they are shared. However, a single party is always the one who pays for the risk - the Owner, in every case. In fact, it is not unusual for the Owner to pay more than once for a particular risk or contingency item. For example, if the particular contingency is poorly defined in the contract documents, the contractor will include a contingency cost in his bid. Subsequently, the contractual arrangements may be such that if the contingency develops into some form of delay, damage or other cost, the contractor will seek to be paid separately for these costs. This represents a second layer of cost for the same contingency and the costs associated with arbitration or litigation may be considered a third layer of cost, all of which is borne finally by the Owner.

The objective of the legal and contractual elements of a tunneling project should be to achieve the minimum cost for the project through the proper sharing of the risks associated with groundwater control. This, then, is the perspective throughout the following discussion.

6.20 CONTRACT DOCUMENTS

6.21 Basic Elements

The set of documents which controls the conditions of bidding and provides the contractual framework for tunnel construction set the context for price determination as well as for the execution of the work. As mentioned in a preceding section, the presence of groundwater always injects an element of uncertainty, especially at the time of bidding. Thus, contingency costs become a large component of the bid price. The contract documents are thus the primary tool for control of contingency costs, especially with regard to groundwater control. The means by which this control can be effected through contract documents include the following:

> Disclosure of all subsurface exploration data, both factual and interpretive. This should be done, whatever the scope of the exploratory and testing programs. The contractor's ability to anticipate groundwater control problems is directly related to the amount of information available to him. Further, the cost of the project is only increased if the contractor must gather his own information in order to duplicate data already obtained, but withheld from bidders, by the Owner.
>
> A clear statement of the responsibility of the contractor for groundwater control should be couched in terms of practical, realistic results to be achieved. Further, there should be a clear definition of the means of measurement by which the achievement of these results will be determined.
>
> There should be a minimum limitation placed on the bidders' ingenuity to produce the specified results.
>
> There should be procedural provisions to exclude unskilled or inadequate consideration of groundwater problems in bidding and to demonstrate in advance of the tunneling that system designs are competent, that system performance is adequate, and that appropriate monitoring will be done as the work progresses.

Payment provisions should allow for fair compensation for groundwater control activities, but without an incentive to unnecessarily expand this element of the work, such as through unit prices for quantity of water or grout pumped or for additional wells.

There should be a change mechanism for accomodating unanticipated aspects of groundwater control. Because monitoring of the dewatering activity can provide early warning of such conditions, it is an important consideration in controlling the adverse effects of changed groundwater circumstances. For these reasons, an owner designed and operated groundwater monitoring system may prove to be a good investment in minimizing these effects.

Whether the change mechanism is through a changed-condition clause, or some other procedural feature of the contract, the contract documents should clearly describe its features. The amount of price increase in bids due to contingencies will be directly influenced by how well the bidders understand how unanticipated groundwater conditions will be identified and how the costs of their resolution will be compensated.

6.22 Execution

Once it is decided how much of the groundwater control design will be specified and how much will be left to the option of the contractor it is necessary to accurately convey this information in the contract documents. The specifications must be written in a way which clearly state what is required of the contractor for performing groundwater control. The specifications should also convey how the performance of this groundwater control system will be measured. By knowing what exactly is required of him and also upon what basis the performance of the groundwater control system will be measured, the contractor will have the best chance of designing a system which will perform as specified.

Sometimes such a partial specification takes the form of the Owner soliciting a lump sum price bid for a fixed number of dewatering wells, with a unit price bid for additional wells as they may be required. The procedure has some merit, but in practice difficulties often develop. In variable soils the number of wells required is often a function of the contractor's skill in adapting well design and construction methods to the conditions encountered, and in selecting the most favorable sites for wells. It is not in the Owner's interest to have given the contractor an incentive to construct as many wells as possible.

An alternative procedure is for the Owner to design and specify a minimum dewatering system that the contractor must install. The responsibility for the adequacy of the system, and the cost of any supplementary effort required, remains with the contractor. The advantages claimed for the method are several. It assures that a reasonable dewatering effort will be made in advance of excavation. In the course of installing the minimum system, an experienced contractor can develop data to help him gage the necessity of supplemental work. The minimum system approach avoids the confusing division of responsibility for groundwater control and it also reduces the possibility that an inexperienced contractor will attempt the work with unsuitable methods.

If part of the groundwater control system is specified, it is necessary that the anticipated performance of the system be accurately described. The contractor must know the extent of groundwater control to be provided by the design system, and how much additional control he is responsible for maintaining. By eliminating any ambiguity and overlap of responsibility, contingencies supporting this uncertainty can be eliminated from the bid price.

When conditions indicate that more than one groundwater control method is capable of providing the satisfactory result, a groundwater control method should not be specified in the contract documents. If a groundwater control method is specified, a statement to the effect that alternate methods of groundwater control are allowed should be included. This allows for ingenuity in groundwater control and also produces the lowest competitive bid. In this case it is necessary to state exactly what the groundwater control system is required to do and also how its performance will be measured. If the contractor is allowed to design his own groundwater control system, full disclosure of all technical information should be contained in the contract documents.

One of the objectives of specifications is to protect the Owner from unrealistic pricing of bids as a result of inadequate or unskilled approaches to groundwater control problems. A procedure which has proven successful in this regard is the two-stage submittal. Prior to beginning work, the contractor submits to the engineer for review a detailed plan of his proposed dewatering system.. Review by the engineer does not relieve the contractor of his responsibility for the adequacy of the system, but it does provide the engineer with an indication of the thoroughness with which the contractor is anticipating the control of groundwater. During the construction of the dewatering system the contractor, in accordance with good practice, will be making observations and conducting tests to evaluate the underground conditions. This information will be much more

complete than that available at the time of the bid and may suggest substantial modification of the final design. Hence, a second submittal is more meaningful. After completion of the dewatering system installation and prior to the start of tunneling, the contractor again submits a detailed plan of the dewatering system as constructed, together with test data and computations demonstrating that the system is capable of achieving the specified result. This second submittal, therefore, forces the discovery of pertinent supplementary information and, through the demonstration of results in advance of tunneling, minimizes the risks of interrupted production and other costs associated with unanticipated groundwater problems.

6.30 ALLOCATION OF RISK

For the purposes of this discussion, risk is defined as the potential for harm, damage, or other adverse effect, all of which may be measured in terms of added cost of the total tunneling project. Avoidance of risk has, in recnt years, become an increasingly popular concept throughout our society and commerce. But some level of risk resides in any undertaking. Construction has been described as the most complex activity routinely undertaken by man. Risk is embodied in the natural enviroment of the construction activity and the human processes involved (principally, communications). The complexity of the construction activity accentuates the risk inherent in communications (disputes, claims, etc.). Tunneling is perhaps the most complex construction activity, occuring in the underground, a high-risk environment. This peculiar concentration of potential risk is a forceful background for considering how to allocate equitable shares of the risk to the participants in a tunneling project.

A summation of the elements of current practice can be organized from this perspective, considering three principals in the tunneling enterprise (Owner, Engineer, Contractor) and dividing the potential risks into three topical categories (environmental or site-related, communications-related, and procedural) as follows:

6.31 Environmental

The site-related risks are generally considered to be the responsibility of the Owner. In other words, the Owner assumes these risks when the tunnel alignment was determined. In groundwater terms, the risks involve the adverse effects on the contractor's cost resulting from unexpected groundwater control problems and from the adverse effects of groundwater control upon third parties. The risk control techniques that have been found to be most effective are: A technically adequate investment in subsurface exploration prior to design and bidding;

full disclosure of the results of subsurface explorations in the bidding documents; and the provision of a change mechanism, such as a changed-condition clause in the contract documents, perhaps with its effectiveness enhanced by an owner-designed monitoring system.

6.32 Communications

Most of the risks associated with disputes over division of responsibility, interpretation of contractual provisions, appropriateness of changes in procedure, etc., are fundamentally manifestations of communication deficiencies. After he has completed his technical design tasks, the Engineer's primary role is that of communicator. Hence, this category of risks is best controlled by the Engineer through such devices as: disclosure of basic design intent and concepts; through proper definitive specifications, as discussed in earlier sections of this report; by review of the technical aspects of the Contractor's groundwater control program so as to insure not only technical adequacy, but the thoroughness of consideration as well; and an effective program of continuous monitoring throughout the project.

6.33 Procedural

This category of risks is most closely related to the Contractor, whose proper role is to assess the groundwater problem and effect solutions within the limits of the data available, and hence, to monitor groundwater conditions so as to anticipate the need for modifications before tunnel progress is affected. Accordingly, the risks generally fall into the category of increased costs associated with delays, and unreimbursable costs. These cost elements are often a second layer of cost added to the contingency prices already included in the bid items in order to accomodate uncertainty in the basic information available at the time of bidding. The techniques available to the Contractor for the control of these risks are as follows: A competent program of exploration, testing, and design so as to develop the least expensive and most flexible groundwater control system; the conscientious pursuit of an adequate program of groundwater monitoring after the initial system has been installed; and the utilization of the contract provisions for changes in compensation in connection with unanticipated groundwater conditions.

From this perspective it is evident that the concept of allocating risks must be based on two equally important concepts; who is in the most effective position to mitigate a particular variety of risk, as well as who is equitably responsible for the costs involved? The advantage of considering the concept from this viewpoint is to make more evident that fact that the process of risk allocation involves responsibility for action

(mitigation) as well as reaction (compensation). Much has been done in consideration of the reactive aspects of the concept. However, it is likely that the total cost to the project would be significantly reduced if equal attention were addressed to the responsibility of the various parties to take action to control risk and to concentrate on providing incentives for such action.

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FEDERALLY COORDINATED PROGRAM (FCP) OF HIGHWAY RESEARCH AND DEVELOPMENT

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The diagonal double stripe on the cover of this report represents a highway and is color-coded to identify the FCP category that the report falls under. A red stripe is used for category 1, dark blue for category 2, light blue for category 3, brown for category 4, gray for category 5, green for categories 6 and 7, and an orange stripe identifies category 0.

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Safety R&D addresses problems associated with the responsibilities of the FHWA under the Highway Safety Act and includes investigation of appropriate design standards, roadside hardware, signing, and physical and scientific data for the formulation of improved safety regulations.

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